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DEPARTMENT OF HIGHWAYS

JOINT HIGHWAY RESEARCH PROJECT

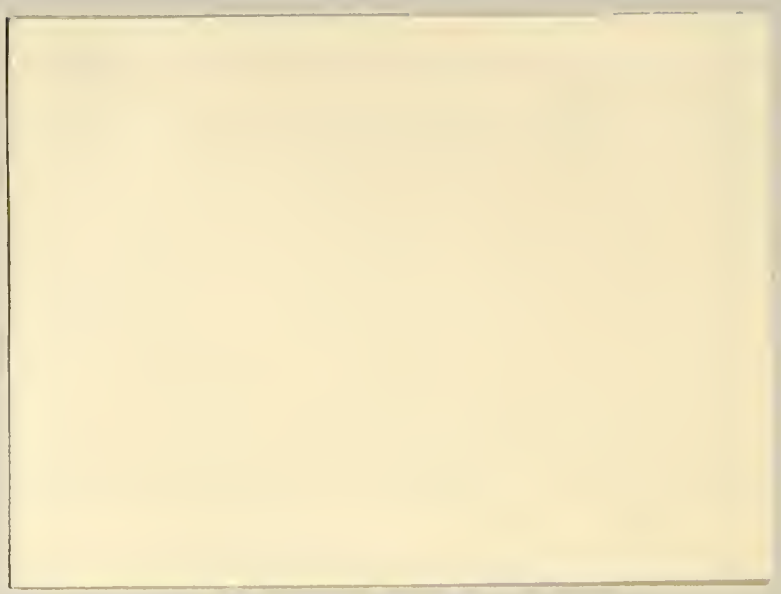
IN/JHRP-83/3

Interim Report

# USE OF PEATS AS EMBANKMENT FOUNDATIONS

H. A. Gruen, Jr.
C. W. Lovell

PURDUE UNIVERSITY



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C. W. Lovell

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Use Of Peats As
Embankment Foundations

To: H.L. Michael, Director
Joint Highway Research Project

February 9, 1983

Project: C-36-5P

From: C.W. Lovell, Research Engineer
Joint Highway Research Project

File: 6-6-16

Attached is an Interim Report on the JHRP study titled "Controlling Movements of Embankments Over Peats and Marls." The report is entitled "Use of Peats as Embankment Foundations" and is authored by H. Allen Gruen, Jr. and C.W. Lovell of our staff.

The report proposes a classification system for Indiana peats, which is in basic conformance with that developed in Committee D18 of the American Society for Testing and Materials. It further proposes sampling, testing, and analysis procedures which will produce reasonable predictions and control of embankment settlements where peat is the foundation.

The report is presented prior to the completion of the total project, so that the Indiana Department of Highways may consider its recommendations for the next construction season. A second and final report will treat the subject of embankments built over Indiana marls.

The report is submitted for review, comment and acceptance in partial fulfillment of the referenced JHRP study.

Respectfully submitted,

C.W. Lovell
Research Engineer

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Interim Report

# USE OF PEATS AS EMBANKMENT FOUNDATIONS

By

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Project No.: C-36-5P
File No.: 6-6-16

Prepared for an Investigation
Conducted by the
Joint Highway Research Project
Purdue University
in Cooperation with the
Indiana Department of Highways

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the material presented herein.

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16. Abstract

Dealing with peat as an engineering material has been avoided in the past whenever possible. This is reflected in the lack of knowledge most engineers have concerning peat for use as a foundation material. This report is presented as an aid to the engineer faced with the problem of building on peat, and should shed light on the unique properties and behavior of this organic material.

The report proposes a classification system for Indiana peats, which is in basic conformance with that developed in Committee D 18 of the American Society for Testing and Materials. It further proposes sampling, testing, and analysis procedures which will produce reasonable predictions and control of embankment settlements where peat is the foundation.

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# TABLE OF CONTENTS


| | Page |
|---|---|
| LIST OF TABLES | ix |
| LIST OF FIGURES | x |
| LIST OF ABBREVIATIONS | xii |
| LIST OF SYMBOLS | xiii |
| HIGHLIGHT SUMMARY | xvi |
| INTRODUCTION | 1 |
| Scope | 3 |
| DISTRIBUTION | 4 |
| United States | 4 |
| Indiana | 6 |
| CLASSIFICATION OF PEAT | 8 |
| Von Post's humification scale | 8 |
| Radforth system | 10 |
| Proposed ASTM system | 12 |
| PHYSICAL PROPERTIES | 17 |
| Fiber content | 17 |
| Water content | 17 |
| Ash content | 19 |
| Organic content | 19 |
| Void ratio | 20 |
| Density of solids | 20 |
| Density | 20 |
| Acidity | 21 |
| Atterburg limits | 21 |
| Permeability | 22 |

MECHANICAL PROPERTIES ..... 24
Shear strength of peat ..... 24
Effect of fibers ..... 24
Other influences ..... 30
Determination of shear strength ..... 31
Shear behavior ..... 33
Shear strength improvement ..... 34
Compressibility of peat ..... 35
One dimensional consolidation ..... 35
Lateral movements ..... 42
Factors affecting compressibility ..... 43
Compressibility of natural deposits .... 48
Summary ..... 49
SAMPLING ..... 52
Disturbed sampling ..... 52
Undisturbed sampling ..... 53
Disturbance effects ..... 55
Fibrous peat ..... 56
Amorphous-granular peat ..... 60
Discussion ..... 62
Summary ..... 63
TESTING ..... 64
Index properties ..... 64
Fiber content ..... 65
Density ..... 66
Determination of pH ..... 66
Compression test ..... 67

Triaxial testing ..... 72

REVIEW OF CONSOLIDATION THEORIES ..... 74

Terzaghi theory ..... 75

Buisman theory ..... 78

Berry and Poskitt theory ..... 79

Gibson and Lo theory ..... 81

Empirical procedures ..... 84

PREDICTING SETTLEMENTS OF PEAT ..... 87

Terzaghi and Buisman methods ..... 87

Berry and Poskitt theory ..... 87

Compression index method ..... 88

Empirical method ..... 89

Gibson and Lo model ..... 91

Applicability to lab data ..... 91

Applicability to field data ..... 93

Varying the stress change term ..... 97

Case study ..... 101

Limitations ..... 104

Summary ..... 106

ROAD CONSTRUCTION METHODS ..... 108

Relocation ..... 108

Replacement ..... 110

Preconsolidation ..... 111

Preloading peat ..... 111

SUMMARY AND RECOMMENDATIONS ..... 114

Need for a uniform classification system .... 114

Physical properties ..... 114

Mechanical properties ..... 115

Sampling and testing ..... 115

Embankment design ..... 118

Preloading and control ..... 119

Concluding remarks ..... 120

Recommendations for future research ..... 121

REFERENCES ..... 122

APPENDICES ..... 132

# LIST OF TABLES


| Table | | Page |
|---|---|---|
| 1. | Classification of peat structure. From MacFarlane (1969). ..... | 11 |
| 2. | Properties designating nine pure coverage classes. From MacFarlane (1969). ..... | 13 |
| 3. | Grouping of organic materials. Tentative ASTM Standard. ..... | 16 |
| 4. | Relative values of various peat properties for predominate types. From MacFarlane (1969). ..... | 18 |
| 5. | Empirical equations for compressibility of peat. ... | 85 |
| 6. | Road construction methods over organic terrain. After MacFarlane (1969). ..... | 109 |
| 7. | Typical ranges of physical properties for peat. ... | 116 |

# LIST OF FIGURES


| Figure | | Page |
|---|---|---|
| 1. | Frequency of occurrence of peat deposits. After Soper and Osbon (1922), and Witczak (1972)..... | 5 |
| 2. | Effect of compression on peat fabric. From Gruen (1982)..... | 25 |
| 3. | Shear strength of peat as a function of effective normal stress. After Helenelund (1975a)... | 27 |
| 4. | Shear failure modes. From Gruen (1982)..... | 29 |
| 5. | Phase diagram for a fibrous Indiana peat..... | 36 |
| 6. | Laboratory strain-time curve for peat. From Dhowian and Edil (1980)..... | 40 |
| 7. | Consolidation response curve. From Dhowian and Edil (1980)..... | 41 |
| 8. | Results from settlement observations of a test embankment on peat. From Helenelund (1975a).... | 44 |
| 9. | Compression index versus water content. From MacFarlane (1969)..... | 46 |
| 10. | Compression index versus void ratio. From MacFarlane (1969)..... | 47 |
| 11. | Results of unconfined compression tests. From Helenelund (1972a)..... | 58 |
| 12. | Variation of water content with effective stress for peat samples. From Helenelund (1975a).... | 59 |
| 13. | Load compression curves from oedometer tests. From Helenelund (1972a)..... | 61 |
| 14. | Comparison of Taylor's method for determining the end of primary consolidation and that obtained by pore-pressure measurement. Step load test, 24 hour duration..... | 69 |
| 15. | Comparison of Taylor's method for determining the end of primary consolidation and that obtained by pore-pressure measurement. Step load test, loading during primary strain only..... | 70 |
| 16. | Results of oedometer tests under various loading sequences..... | 71 |

17. The Gibson and Lo model..... 82

18. Predicted and observed compression of peat under a pressure increment of 200 – 400 kPa. After Edil and Dhowian (1979)..... 92

19. Settlement data under preload. After Edil (1981)..... 94

20. Loading sequence for field case..... 95

21. Applicability of model to field loading case..... 96

22. Use of model to predict settlement behavior of 12 kPa load using parameters from test run at 25 kPa..... 99

23. Use of model to predict settlement behavior of 12 and 25 kPa load using parameters from test run at 56.91 kPa..... 102

24. Longitudinal subsurface section. From Edil (1981)... 103

25. Theoretical predictions and field settlement data. From Edil (1981)..... 105

# LIST OF ABBREVIATIONS

| | |
|---|---|
| AASHTO | American Association of State Highway and Transportation Officials |
| ACSSM | Association Committee on Soil and Snow Mechanics |
| ASCE | American Society of Civil Engineers |
| ASTM | American Society for Testing and Materials |
| NRC | National Research Council |
| SMFE | Soil Mechanics and Foundation Engineering |
| USCS | Unified Soil Classification System |
| WHC | water holding capacity |

# LIST OF SYMBOLS

| | |
|---|---|
| a | - rheological parameter |
| $a_v$ | - coefficient of compressibility |
| $A_c$ | - ash content |
| b | - rheological parameter |
| c' | - effective stress strength intercept |
| cc | - cubic centimeter |
| cm | - centimeter |
| C | - line intercept |
| $C_c$ | - compression index |
| $c_v$ | - coefficient of consolidation |
| D | - slope of the line |
| $e_o$ | - initial void ratio |
| ft | - feet |
| g | - gram |
| G | - density of solids |
| $h_o$ | - initial thickness |
| H | - layer thickness after primary consolidation |
| $H_o$ | - initial thickness |
| i | - exponential parameter |
| in. | - inch |
| k | - coefficient of permeability |
| kg | - kilogram |
| kPa | - kilo-Pascal |
| LIR | - load increment ratio |
| m | - meter |
| max | - maximum |

mm - millimeter

MPa - mega-Pascal

$p_o'$ - initial effective overburden pressure

sec - second

sq - square

S - total settlement

$S_o$ - primary settlement

$S_s$ - secondary settlement

t - time

$t_a$ - time at which secondary strain begins

$t_k$ - time at which tertiary strain begins

$t_o$ - time to complete primary strain

u - excess of equilibrium pore water pressure

vs - versus

$w_n$ - natural water content

Y - strain rate

z - distance from the element to the drainage surface

$\alpha_p$ - coefficient of primary consolidation

$\alpha_s$ - coefficient of secondary compression

$\Delta e$ - change in void ratio

$\Delta p$ - applied stress change

$\Delta \sigma'$ - change in effective stress

$\Delta \sigma_I$ - stress level at which prediction is to be made

$\Delta \sigma_{II}$ - stress level for which rheological parameters were calculated

$\epsilon_i$ - instantaneous strain

$\epsilon_p$ - primary strain

$\epsilon_s$ - secondary strain

$\epsilon_t$ - tertiary strain

$\epsilon(t)$ - strain as a function of time

$\epsilon_v$ - vertical strain

$\rho_w$ - density of water

$\lambda$ - rheological parameter

$\phi'$ - effective angle of stress strength

$\rho_d$ - dry density

$\sigma'$ - effective normal stress

$\sigma_1$ - major principal stress

$\sigma_3$ - minor principal stress

$\tau$ - shear stress

$\tau_f$ - shear strength

$>$ - greater than

$<$ - less than

## HIGHLIGHT SUMMARY

This report is presented as an aid to the engineer faced with the problem of dealing with peat. Peat is abundant in many parts of the world; however, few engineers know more than the fact that peat is not a good foundation material and should be avoided whenever possible. This report reviews the physical and mechanical properties of peat, methods of sampling and test procedures used to determine properties. The unique characteristics of peat are presented and classification systems which distinguish peat from other organic soils are outlined. A large portion of this report centers on the stress-deformation behavior of peat and methods used to estimate field settlements. Methods of building embankments over peat for use as highway foundations are briefly covered. In many cases preloading the peat to improve its strength is a desirable way of utilizing peat as a foundation material. A rheological model first presented by Gibson and Lo (1961) is applied to peat for use as a control over preloading duration. The applicability of the model is tested on both laboratory and field data and is found to model actual peat behavior quite accurately.

# Introduction

The highly compressible nature of peat makes it one of the most undesirable foundation materials for highway construction. Besides its high compressibility, peat in a natural state has very low strength, which adds to its reputation as a poor foundation material. Highway engineers try to avoid peat deposits whenever possible. Local peat pockets and shallow deposits are generally excavated and replaced by a more desirable material when encountered. However, there are situations when a peat deposit cannot be avoided. If the deposit is of sufficient depth and extent it may not be economical to excavate and replace the peat. In such a situation the highway engineer may have to use the peat as a foundation material. When a highway alignment must pass over a peat deposit, the load caused by the pavement and subgrade will cause some settlement to occur. For this reason, highway pavements must be elevated above the peat deposit by means of an embankment. This embankment causes an additional load on the peat resulting in more settlement. Predicting and dealing with these settlements has been a problem for highway designers and foundation engineers.

Even though peat is an abundant material in many parts of the world, most geotechnical engineers are not

sufficiently familiar with the properties and behavior of peat to deal effectively with it. Peat materials present many unique problems which are not encountered with mineral soils. Due to this unusual behavior, specialized procedures and methods of analysis must be used when dealing with peat.

One of the major problems enginers have had with peat is relating its properties and behavior in a systematic way. This problem is caused mainly by the lack of a universal classification system which adequately groups peat and highly organic soils. The Unified Soil Classification System (USCS), originally developed by Casagrande (1948), groups all highly organic soils into one category called peat, which is identified by "color, odor, spongy feel, and frequently by fibrous texture" (Holtz and Kovacs, 1981). The AASHTO (1970) classification system treats peat in a similar manner by placing all peats and highly organic soils into one group designated A-8, which is given a subgrade rating of unsatisfactory. Trying to deal with peat, its properties, or behavior in a systematic way requires a more complete classification system. Considering all peats and highly organic soils in one group would be like treating all fine grained soils as one material with the same properties and behavior. Landva and LaRochelle (1982) stress that "it is important to dis-

tinguish between the very large variety of materials within the group of soils currently referred to as peat. The properties of these materials vary from that of textile-like fabric to that of a jelly-like substance. A case record of construction on peat land is of little value without such distinction and without a detailed description of the peat involved." Classification of peat and highly organic materials along with the properties and behavior of peat will be dealt with in later sections.

Scope. This report is written as an aid to the engineer faced with the potential problem of having to deal with peat as a foundation material. The main emphasis is on compression behavior and settlement prediction of peat; however, many other aspects such as the physical characteristics, sampling, testing and classification of peat materials are discussed. From the coverage of these topics the foundation engineer can establish a plan of testing, interpret the relevant data, and predict the settlement and strength characteristics of peat for use as a highway embankment foundation.

## DISTRIBUTION

Peat is abundant in many countries around the world. Russia, Ireland, Norway and Canada are known for their extensive peat deposits. In the United States, the large peat deposits may be roughly assigned to two general regions--the northern or glacial, and the Gulf and Atlantic coastal. Figure 1 shows the approximate distribution of peat deposits in the contiguous 48 states.

United States. The northern peat region, which contains the most extensive deposits in the United States, includes Minnesota, Wisconsin, Michigan, eastern South Dakota, the northern parts of Iowa, Illinois, Indiana, and Pennsylvania, New York, New Jersey, and the New England states. This region is characterized by numerous ponds, marshes, and lakes formed by glacial action during Pleistocene time and by relatively low temperature and high humidity during the growing season. Most of the peat originates in basins. Probably the largest peat deposit in this country is in northern Minnesota. It covers nearly 10,000 square kilometers (2,500,000 acres) (Soper and Osbon, 1922).

The Atlantic Coastal region embraces the southern part of Delaware, the eastern parts of Delaware, Maryland, Virginia, North Carolina, South Carolina, and

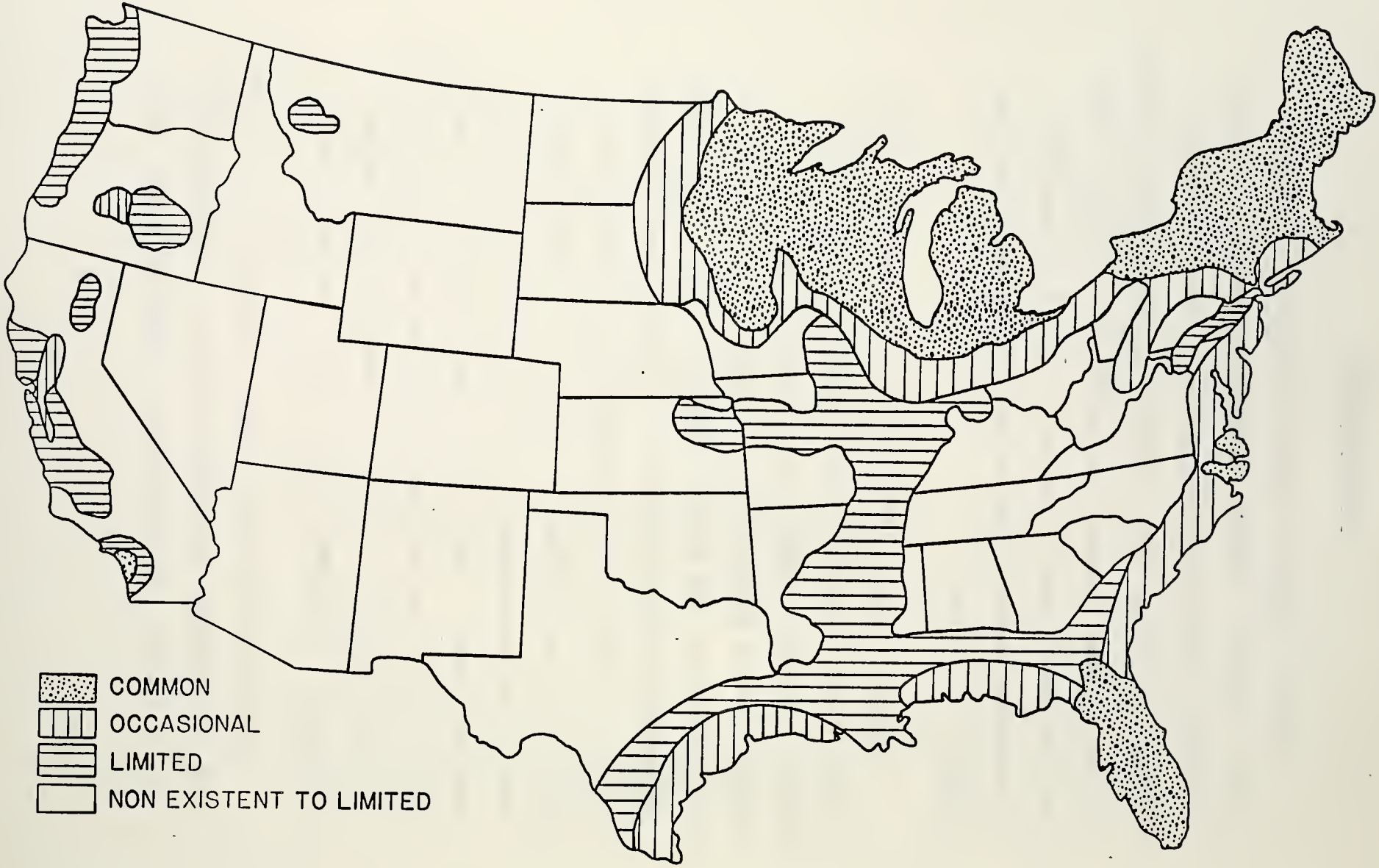



Figure 1. Frequency of occurrence of peat deposits. After Soper and Osbon (1922), and Witczak (1972).

Georgia, and all of Florida. The nearness of the ocean causes heavy rainfall and high relative humidity in this region, and the deposits occur in drowned valleys and lagoons, which were formed by the gradual emergence of the Coastal Plain and by wave action on the flat, imperfectly drained areas farther inland.

Peat also occurs in a belt of land adjoining the Gulf Coast which includes parts of Alabama, Mississippi, Louisiana, and Texas. In the west, peat is found in the valleys of Sacramento and San Joaquin Rivers of California, and in the basins of several lakes and rivers in Oregon and Washington. Peat also occurs to a limited extent along the Mississippi River.

Indiana. The major peat deposits of Indiana are confined to the glaciated region in the northern part of the state. The greater part of the peat occupies the basins of ancient glacial lakes, but a few deposits were formed in depressions between sand dunes and in marshes along meandering streams. The following counties contain most of the peat in Indiana: Allen, Dekalb, Elkhart, Fulton, Jasper, Lagrange, Lake, Laporte, Marshall, Newton, Noble, Porter, Pulaski, St. Joseph, Starke, Steuben, Wabash, and Whitley. Indiana's peat deposits range from 0.02 to 10 square kilometers (5 to 2,500 acres) in area and extend to a maximum thickness of about 20 meters (70 feet);

however, few deposits exceed 3 meters (10 feet) (Taylor 1907). A large quantity of peat is underlain by soft marl, which is also a poor foundation material. The principal types of peat found in Indiana are sphagnum-moss peat and reed-sedge peat.

A comprehensive survey of peat in Indiana was made by A. E. Taylor. Taylor (1907) described the peat deposits in 21 northern counties, showing areas underlain by peat, locations of deposits, and numerous thicknesses. Taylor presents this information on 20 county maps and numerous, more detailed maps of larger scale.

## CLASSIFICATION OF PEAT

Peat is used as an energy resource, for horticultural purposes and observed as a botanical substance, in addition to being used as an engineering material (Decker, 1982). Each use has resulted in a classification scheme which reflects the properties of interest. Peat is classified according to botanical composition. Floristic designations such as sphagnum and sedge peats are sometimes used. Peat has been classified for botanical purposes according to the type of plant growth it can support. Peat is classified for horticultural purposes based on its nutrient composition. The energy industry classifies peats according to combustibility, energy content, and ash content. Clearly there are numerous ways to classify peat, depending on its intended use. It is beyond the scope of this text to cover all the existing systems used to classify peat, however, the classification systems which have some use to the geotechnical engineer will be presented.

Von Post's humification scale. The Von Post (1922) classification system is widely used in horticulture, agriculture, forestry, as well as the energy industry. This system was developed in connection with a major inventory of peat as a growing medium. It is based on plant genera, modes of formation, chemical and

mineral contents and vegetation supported. The Von Post system is best known to the soils engineer for its classification of peat based on decomposition. This method of classification is observational and intended for field use. The degree of humification, designated by the letter 'H', is subdivided into 10 categories, as follows (Radforth and Brawner, 1977).

$H_1$ Completely undecomposed peat which, when squeezed, releases almost clear water. Plant remains easily identifiable. No amorphous material present.

$H_2$ Almost completely undecomposed peat which, when squeezed, releases clear or yellowish water. Plant remains still easily identifiable. No amorphous material present.

$H_3$ Very slightly decomposed peat which, when squeezed, releases muddy brown water, but for which no peat passes between the fingers. Plant remains still identifiable, and no amorphous material present.

$H_4$ Slightly decomposed peat which, when squeezed, releases very muddy dark water. No peat is passed between the fingers but the plant remains slightly pasty and has lost some of the identifiable features.

$H_5$ Moderately decomposed peat which, when squeezed, releases very "muddy" water with also a very small amount of amorphous granular peat escaping between the fingers. The structure of plant remains is quite indistinct although it is still possible to recognize certain features. The residue is strongly pasty.

$H_6$ Moderately strongly decomposed peat with a very indistinct plant structure. When squeezed, about one-third of the peat escapes between the fingers. The residue is strongly pasty but shows the plant structure more distinctly then before squeezing.

$H_7$ Strongly decomposed peat. Contains a lot of amorphous material with very faintly recognizable plant structure. When squeezed, about one-half of the peat escapes between the fingers. The water,

if any is released, is very dark and almost pasty.

$H_8$ Very strongly decomposed peat with a large quantity of amorphous material and very indistinct plant structure. When squeezed, about two-thirds of the peat escapes between the fingers. A small quantity of pasty water may be released. The plant material remaining in the hand consists of residues such as roots and fibers that resist decomposition.

$H_9$ Practically fully decomposed peat in which there is hardly any recognizable plant structure. When squeezed, almost all the peat escapes between the fingers as a fairly uniform paste.

$H_{10}$ Completely decomposed peat with no discernible plant structure. When squeezed, all the wet peat escapes between the fingers.

Radforth system. Another major classification system is that proposed by Radforth (1952). Radforth peats by definition contain less than 25 percent mineral content by weight. Peats in this system are classified according to structure and vegetative cover. The structural classification ranges from granular to fibrous. Granular peats consist of highly decomposed organic matter which have mechanical properties similar to mineral soils. These peats are referred to as amorphous-granular. At the other extreme, peat may be composed primarily of plant fibers which show little decomposition. These are called fibrous peats. Fibrous peats are subdivided into fine-fibrous and coarse-fibrous depending on the nature of the organic fibers. The Radforth structural classification system is shown in Table 1. The vegetative cover which the

TABLE 1. Classification of peat structure.
From MacFarlane (1969).

| Predominate Characteristic | Category | Name |
|---|---|---|
| Amorphous-granular | 1 | Amorphous-granular peat. |
| | 2 | Non-woody, fine-fibrous peat. |
| | 3 | Amorphous-granular peat containing non-woody fine fibers. |
| | 4 | Amorphous-granular peat containing woody fine fibers. |
| | 5 | Peat, predominately amorphous-granular, containing non-woody fine fibers, held in a woody, fine-fibrous framework. |
| | 6 | Peat, predominantly amorphous-granular containing woody fine fibers, held in a woody, coarse-fibrous framework. |
| | 7 | Alternate layering of non-woody, fine-fibrous peat and amorphous-granular peat containing non-woody fine fibers. |
| Fine-fibrous | 8 | Non-woody, fine-fibrous peat containing a mound of coarse fibers. |
| | 9 | Woody, fine-fibrous peat held in a woody, coarse-fibrous framework. |
| | 10 | Woody particles held in non-woody, fine-fibrous peat. |
| | 11 | Woody and non-woody particles held in fine-fibrous peat. |
| Coarse-fibrous | 12 | Woody, coarse |
| | 13 | Coarse fibers criss-crossing fine-fibrous peat. |
| | 14 | Non-woody and woody fine-fibrous peat held in a coarse-fibrous framework. |
| | 15 | Woody mesh of fibers and particles enclosing amorphous-granular peat containing fine fibers. |
| | 16 | Woody, coarse-fibrous peat containing scattered woody chunks. |
| | 17 | Mesh of closely applied logs and roots enclosing woody coarse-fibrous peat with woody chunks. |

peat can support often infers information about the underlying deposit (MacFarlane, 1958). Radforth's classification of the vegetative cover is shown in Table 2.

Proposed ASTM system. The American Society for Testing and Materials (ASTM) is currently working on a standard classification system which would apply to all interested disciplines. The activity is within Committee D18 (Soils and Rocks), and specifically, Subcommittee D18.18 (Peats and Organic Soils). According to Cohen (1981), "The purpose of the system is to standardize the naming of peat materials so that the peat-producer can better market his product and that the peat-consumer can better select peat materials to fit his requirements. The parameters selected for use in this standard are ones which have been determined to relate to the agricultural, horticultural, geotechnical, and energy uses of peats." The proposed classification would be based on the following properties with the subheadings used as descriptors. Standards are from ASTM (1981).

1. Fiber Content (method of definition not yet established).
    a. Fibric - peat with > 66 2/3% fibers.
    b. Hemic - peat with between 33 1/3% and 66 2/3% fibers.
    c. Sapric - peat with < 33 1/3% fibers.
2. Ash Content (ASTM standard D2974).

TABLE 2. Properties designating nine pure coverage classes. From MacFarlane (1969).

| Coverage type (class) | Woodiness vs. non-woodiness | Stature (approximate height) | Texture (where required) | Growth habit |
|---|---|---|---|---|
| A | Woody | 15 ft. or over | -- | Tree form. |
| B | Woody | 5-15 ft. | -- | Young or dwarfed tree or bush. |
| C | Non-woody | 2-5 ft. | -- | Tall, grasslike. |
| D | Woody | 2-5 ft. | -- | Tall shrub or very dwarfed tree. |
| E | Woody | Up to 2 ft. | -- | Low shrub. |
| F | Non-woody | Up to 2 ft. | -- | Mats, clumps, or patches, sometimes touching. |
| G | Non-woody | Up to 2 ft. | -- | Singly or loose association. |
| H | Non-woody | Up to 4 in. | Leathery to crisp. | Mostly continuous mats. |
| I | Non-woody | Up to 4 in. | Soft or velvety | Often continuous mats, sometimes in hummocks. |

a. Low ash - peat with less than 5% ash.

b. Medium ash - peat with between 5% and < 15% ash.

c. High ash - peat with between 15% and 25% ash.

3. Acidity (ASTM Standard D2976).

a. Highly acidic - peat with a pH less than 4.5.

b. Moderately acidic - peat with a a pH between 4.5 and < 5.5.

c. Slightly acidic - peat with a pH between 5.5 and 7.0.

d. Basic - peat with a pH greater than 7.0.

4. Absorbency (ASTM Standard D2980).

a. Highly absorbent - peat with a WHC (water-holding capacity) greater than 1500%.

b. Moderately absorbent - peat with a WHC between 800 and 1500%.

c. Slightly absorbent - peat with a WHC between 300 and < 800%.

d. Nonabsorbent - peat with a WHC less than 300%.

5. Botanical Composition (Floristic Designation). Name dominant plants in the fibers.

Disagreements as to the behavior of "peat," as evident from a review of the literature, generally can be shown to arise from a lack of proper definition of the material concerned. The term peat has been incorrectly used to describe organic silts and clays with mineral contents as high as 90 percent (Jeffries, 1936). It is important, from a geotechnical point of view, to distinguish between peats and organic soils,

so the described behaviors can be related to the proper material. The ASTM Subcommittee D18.18, in an attempt to distinguish peat from organic soils has proposed the following organic soil classification to be used as a standard definition (See table 3).

1. Peat has less than 25% inorganic material as defined by ASTM Standard D2974.
2. Muck has between 25% and 75% inorganic material.
3. Organic silt or clay has greater than 75% inorganic material (25% or less organics).

Peat is distinguished from phytogenic material of higher rank (i.e. lignite coal) by its lower BTU value on an "as-received," water-saturated basis (see ASTM Standard D388).

Table 3. Grouping of organic materials.
Tentative ASTM Standard.

| Ash Content % | Material Description |
|---|---|
| 0 to < 25 | PEAT |
| 25 to 50 | peaty muck |
| 25 to 75 | MUCK |
| 50 to 75 | silty or clayey muck |
| > 75 | ORGANIC SILT OR CLAY |
| 75 to 90 | highly organic silt or clay |
| 90 to 100 | slightly organic silt or clay |

## PHYSICAL PROPERTIES

Peat is a mixture of fragmented organic material derived from vegetation which has accumulated in wet areas such as swamps, marshes, or bogs. Peat may consist mainly of fibers (fibrous peat), or may tend to be more granular (amorphous-granular peat). Many engineers do not consider peat a soil due to its high organic content. This may be justified when one observes the behavior and characteristics of peat as opposed to a mineral soil. In this section, some physical properties of peat which are of interest to the engineer will be reviewed. The test methods will be presented in a later section.

Fiber Content. The amount of fiber material present seems to have an influence on the mechanical and physical properties of peat. The fiber content is determined by a wet sieving procedure (see Testing section). Amorphous-granular peats tend to behave similarly to mineral soils, whereas this behavior deviates more and more as the fiber content increases. The effect of fibrosity on peat characteristics is shown in Table 4.

Water Content. Peat has a high water holding capacity. Peat with a small amount of mineral contamination (pure peat), generally has a water content

TABLE 4. Relative values of various peat properties for predominant types. From MacFarlane (1969).

| Predominant structural characteristic | Peat Property* | | | | | | |
|---|---|---|---|---|---|---|---|
| | Water content | Natural permeability | Natural void ratio | Natural unit weight | Shear strength | Tensile strength | Compressibility |
| Amorphous-granular | 3 | 3 | 2 | 1 | 3 | 3 | 2 |
| Fine-fibrous (woody and non-woody) | 1 | 2 | 1 | 3 | 2 | 2 | 1 |
| Coarse-fibrous (woody) | 2 | 1 | 3 | 2 | 1 | 1 | 3 |

* On relative scale, 1 is greatest, 3 is least.

varying between 750 and 1500 percent (Feustel and Byers, 1930). Fibrous peat normally has a higher water content than amorphous granular peat. This is because the fibers have an open cellular structure which allows water retention within these organic "solids." The water content is determined for peat in the same manner as for mineral soils.

Ash Content. The ash content is the amount of nonorganic material, expressed as a percentage of dry weight, remaining after a peat sample has been fired at 550 degrees Celcius. This firing process burns off the organic matter which is generally combustible carbonaceous matter. The ash which remains represents the mineral constituent. Pure peats may have an ash content as low as 2 percent, on the other extreme, peat by definition may not have more than 25 percent nonorganic material (Goodman and Lee, 1962).

Organic Content. The organic content of peat has a considerable effect on the physical and mechanical properties of peat. In general, the greater the organic content the greater the water content, void ratio, and compressibility of the peat. The approximate organic content is found by subtracting the ash content from 100 percent. This is only approximate since during the firing process, more than just the organics are burned off. According to Jackson (1958)

this method can produce an error of 5 to 15 percent. MacFarlane and Allen (1964) presented a more accurate method to determine the organic content of peat. This method involves treating the peat sample with chromic acid in hot sulphuric acid. Next, the excess chromic acid that remains after oxidation of the carbon is quantitatively determined by titrating against a standard ferrous ammonium sulphate solution. The ash content method is generally preferred for engineering purposes.

Void Ratio. The void ratio of peats is generally quite high, with fibrous peats greater than amorphous-granular peats. Extreme ranges in void ratio for peats have been reported from 25 to 2, with 5 to 15 being a more usual range (Hanrahan, 1954). The high void ratio corresponds to high water content, high permeability and high compressibility.

Density of Solids. The density of solids of peat depends on the amount of mineral matter present. Values of density of solids for peat vary from 1.1 to 2.5. Most peats are in the range from 1.4 to 1.8 (Wyld, 1956).

Density. The natural density is dependent on the composition of the peat deposit and the water content. The density of pure peat is generally close to that of

water. The density increases as more mineral soil becomes intermixed with the organic matter. MacFarlane (1969) reports natural densities have been observed to range from 0.4 g per cubic cm (25.0 pounds per cubic foot) for a moss peat to 1.2 g per cubic cm (75.0 pounds per cubic foot) for an amorphous-granular peat. Dry densities range from 0.08 g per cubic cm (5 pounds per cubic foot) to 0.32 g per cubic cm (20 pounds per cubic foot), the latter value representing considerable mineral soil contamination.

Acidity. The acidity of a peat deposit is important to the engineer because of the potential corrosive problems which may occur. An acidic peat deposit may have detrimental effects on concrete and metal which come in contact with the deposit. Values of pH have been reported as low as 2.0 and as high as 8.0. The normal pH of peats range from 4 to 7 (Lea 1956).

Atterburg Limits. The consistency limits are difficult if not impossible to obtain on most peats. The fibrous nature of many peats renders the Atterburg limits tests practically useless. The liquid and plastic limits can be determined on highly decomposed and amorphous-granular peats, but the use of this information is questionable. When dealing with peats, determination of the Atterburg limits in general is neither beneficial nor recommended.

Permeability. The permeability of peats varies widely, depending on: (i) the amount of mineral matter present in the peat, (ii) the degree of consolidation, and (iii) the extent of decomposition. Amorphous granular peats tend to be less permeable than fibrous peats. The fabric of fibrous peats provides many interconnected flow channels through which water can easily flow. These channels tend to have a horizontal orientation, causing permeability in the horizontal direction to be higher than that in the vertical direction. At a given void ratio, the horizontal permeability is about 300 times larger than the vertical permeability (Dhowian and Edil, 1980). As peat is compressed, the decrease in void ratio results in large decreases in permeability. Hanrahan (1954) applied a load of 0.56 kg per sq cm (8 pounds per square inch) on a sample of partly humified peat with a natural void ratio of 12 and initial permeability of $4 \times 10^{-4}$ cm per second. After two days, the void ratio was reduced to 6.75 and the permeability to $2 \times 10^{-6}$ cm per second. After seven months under the same load, the void ratio was reduced to 4.5 and the permeability to $8 \times 10^{-9}$ cm per second. The final permeability corresponded to 1/50,000 of the initial permeability value.

Most naturally occurring peats have relatively high initial permeabilities ($1 \times 10^{-3}$ to $1 \times 10^{-5}$ cm

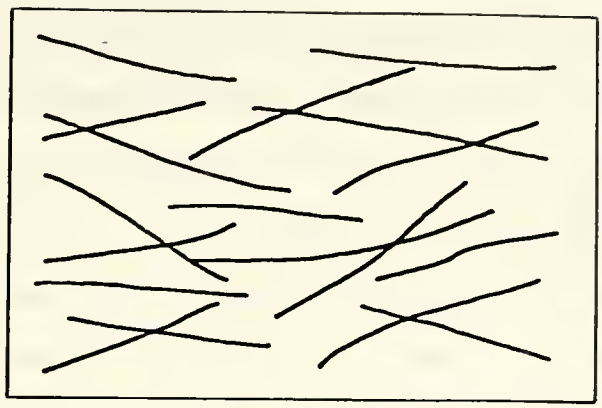
per second) (Wyld, 1956). It should be noted that many peat types are, in fact, relatively impermeable. This is evident by the fact that peat has been used for the impermeable core of rock dams in Norway (Tveiten 1956, Silburn 1972). This extreme range in void ratios makes it difficult to assess the mechanical behavior of peat.

## Mechanical Properties

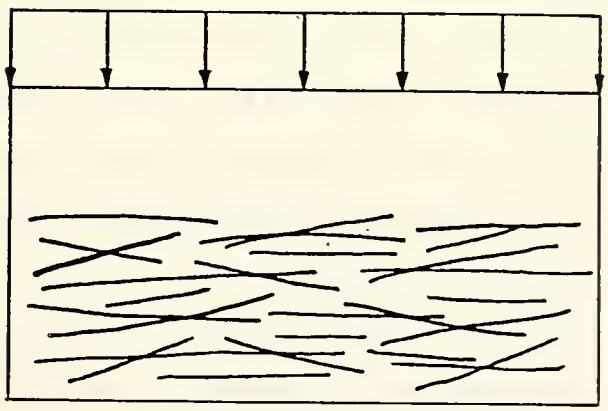
Shear Strength of Peat The shear strength of peat is an important parameter in the analysis of its response to load. Practical situations in which shear strength needs to be assessed are the stability of natural slopes and embankments, evaluation of bearing capacity, and problems involving trafficability. Peat soils generally have a low shear strength and consequently a low bearing capacity in their natural condition. This low strength is undesirable from an engineering viewpoint. Before the shear characteristics of peat can be determined or improved, the mechanism of how shear strength is mobilized in peats should be understood.

Effect of fibers. The fiber content of peat has an important effect on the shear behavior. Peat with a negligible fiber content (amorphous-granular) derives its shear strength in a similar manner to mineral soils. The shear strength is developed from cohesion between the particles and from the frictional resistance between the grains.

Fibrous peat on the other hand mobilizes its shear strength in an entirely different manner. In the natural peat fabric, the organic fibers tend to be somewhat oriented and overlapping (see Figure 2a).



(a) Loose



(b) Compressed

Figure 2. Effect of compression on peat fabric. From Gruen (1982).

In a loose natural state, the fibers are surrounded by water, and the soil matrix has a low shear strength. Most of the shear strength in this condition is from apparent cohesion due to mineral soil (clay) contamination and/or entanglement of the fibers.

During compression the fibers tend to align themselves at right angles to the direction of the applied (vertical) stress. Water is expelled and the fibers come in contact (see Figure 2b). The fibers in this condition act as reinforcement to triaxial shear and the shear strength is a function of the friction between the fibers and the tensile strength of the fibers. In this compressed state, the shear strength is derived from the fiber resistance (frictional component), and the apparent cohesion is relatively small. Thus, as the soil is stressed, the organic fibers become more oriented and move closer together. This results in a large increase in shear strength from the fiber reinforcing effect.

This mechanism of internal resistance seems to agree with Amaryan (1972), who described the behavior of a fibrous peat under a range of loadings. According to Amaryan, the angle of friction is small for low stress increases and the shear strength is mainly due to cohesion (Figure 3). At higher stress levels the

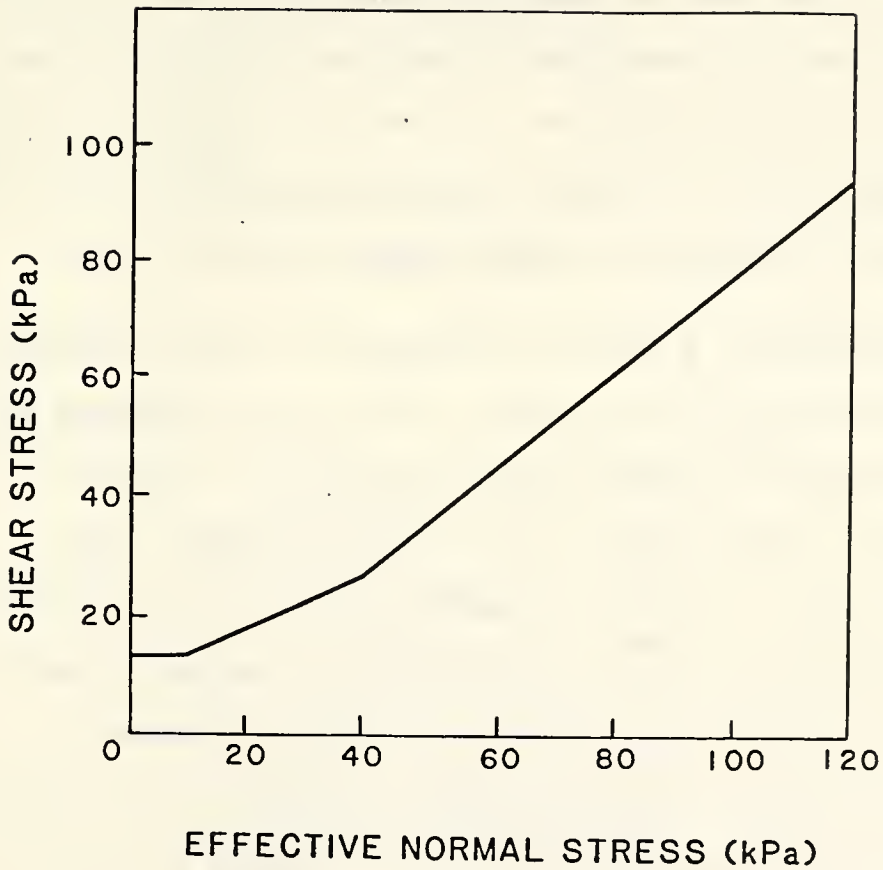
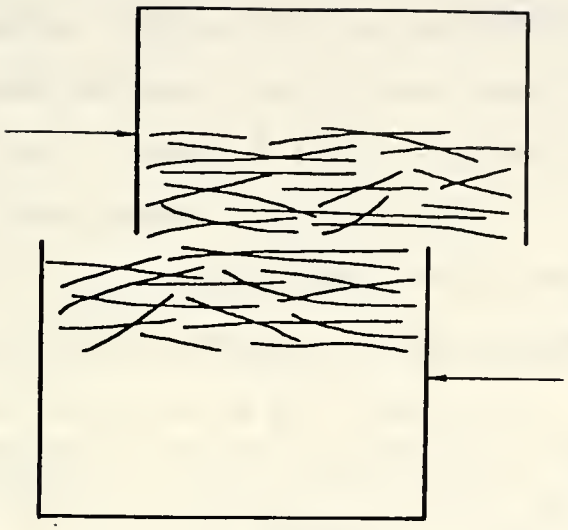



FIGURE 3. Shear strength of peat as a function of effective normal stress. After Helenelund (1975a).

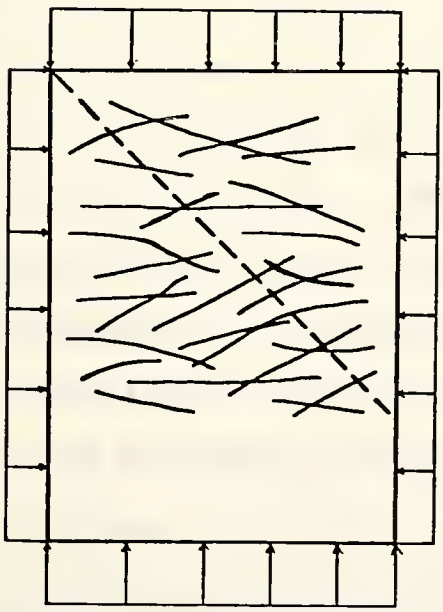
shear strength develops mainly from friction, and the effective cohesion becomes negligible.

It has been shown by many investigators (Helenelund 1975a, Landva 1980a, MacFarlane 1969) that the shear strength of fibrous peat is less in the horizontal plane than in vertical planes. This is to be expected because when peat is sheared parallel to the fibers (horizontally), the reinforcement effect of the overlapping fibers is lost, and the soil will fail with the fibers sliding over each other (see Figure 4a).

In general, a large discrepancy in shear strength is observed when direct shear results are compared with triaxial shear strength (see figure 4b). This is because as noted above, that in direct shear, the failure plane is forced parallel to the fibers, and the reinforcing effect is lost. The actual mode of deformation of a peat deposit under a road embankment or other structure resembles that in triaxial compression. Hence, when determining the shear strength of peat for an embankment or structural loading situation, the triaxial compression test would be the best model. However, if the peat is exposed to a horizontal load and failure is expected to occur on a plain parallel to the fibers, the direct shear test should be used to determine the shear strength.



(a) Direct Shear.



(b) Triaxial Shear.

Figure 4. Shear failure modes.
From Gruen (1982).

Since the mobilization of shear strength is affected significantly by fiber action, as discussed above, any variability in fiber type and content will result in variations in shear behavior. For highly fibrous peats, the effect of the fibers will be quite dominant, to the extent that failure may not be reached in triaxial compression, unless the fibers themselves fail in shear. For peats with a low fiber content, the effect of fiber reinforcement will be insignificant and a shear failure may be expected to occur in the matrix, more or less independent of the fibers, in a manner similar to mineral soils.

Other influences. Other factors have been found to have an influence on the shear strength of peat. According to Wyld (1956), "Qualitatively, the shear strength of peat varies inversely with its water content and directly with its ash content and degree of deformation in compression." Helenelund (1975a) writes "The undrained shear strength diminishes with increasing water content, and an increasing degree of humification."

Shear behavior also depends on permeability. As peat is sheared, excess pore water pressure is generated. The permeability of the peat governs the rate of pore pressure dissipation. Thus, the shear strength varies directly with the permeability. The lower the

permeability, the lower the shear strength. In general, the permeability decreases with increasing degree of humification. Since the degree of humification often varies both horizontally and vertically, in a somewhat erratic pattern, the permeability and shear characteristics will vary accordingly. This makes the determination of a representative shear strength for a particular deposit difficult to ascertain.

Determination of shear strength. Many researchers have tried to quantitatively relate shear strength to some physical properties, with little success. Landva (1980a) states, "In general, it must be concluded that no reliable empirical relationships have as of yet been developed with respect to the shear strength of peat." The undrained shear strength of peat has been measured by field vane tests, although other types of in-situ tests, such as cone penetration and screw-auger pulling have been employed (Huang, 1982). In peats with high degrees of humification, the vane test can be regarded as a reliable shear test. However the vane shear test is not recommended for fibrous peats. Landva (1980b) devotes an entire paper to the topic of using the shear vane test on peat, and concludes that the vane shear test is not a reliable method of determining the in-situ shear strength of peat. Some of the disadvantages of the vane shear test are: (1) vane tests tend to

overestimate actual shear strength, thus being non-conservative, (2) the value of shear resistance is a function of the vane size; larger vanes give lowest values of shear resistance, and (3) when the vane is inserted into the peat, drainage occurs rapidly, resulting in compression and an apparent high value of shear resistance. In addition, the great majority of practical situations require values for shear parameters that will reflect the change in strength with variations in stress environment and loading history. The shear vane test can not simulate either of these situations.

As stated earlier, the actual mode of deformation of a peat deposit under a road embankment or other vertically loaded structure resembles that in triaxial compression. Thus when these types of loads are applied to a peat deposit, the triaxial compression test should be used to determine the shear strength. Many investigators have successfully used both drained and undrained triaxial tests to observe the shear behavior of peat (Gautschi 1965, Adams 1961, Hanrahan et. al. 1967, Hollingshead and Raymond 1972). In most cases large strains were reached before failure occurred.

Hanrahan et. al. (1967), studied the shear behavior of peat and found that the Hvorslev failure

criterion was applicable in peat at water contents ranging from 250 to 350 %. However, most natural peats have water contents much higher than this, so determination of the Hvorslev strength parameters would not be practical for use with peat. Hanrahan et. al. (1967) did conclude however, that the Coulomb - Terzaghi failure criterion could be applied to peats with satisfactory results. The shear strength $\tau_f$ of peat may be represented as:

$$\tau_f = c' + \sigma' \tan \phi' \qquad (1)$$

where $c'$ is the effective stress strength intercept, $\sigma'$ is the effective normal stress, and $\phi'$ is the effective angle of shear strength. Likewise, direct shear tests may be run if the expected failure surface is parallel to the fibers. At present these two laboratory tests run on representative samples give the most reasonable results for stability analysis.

Shear behavior. Test results indicate that the frictional component of shear strength is exceptionally high as compared to that for most inorganic soils. Adams (1961) observed from consolidated, undrained triaxial tests conducted on fibrous peat, that the shear strength is greatly enhanced by consolidation. Adams found the effective angle of shear strength to be as high as 50 degrees. MacFarlane (1969) writes "all

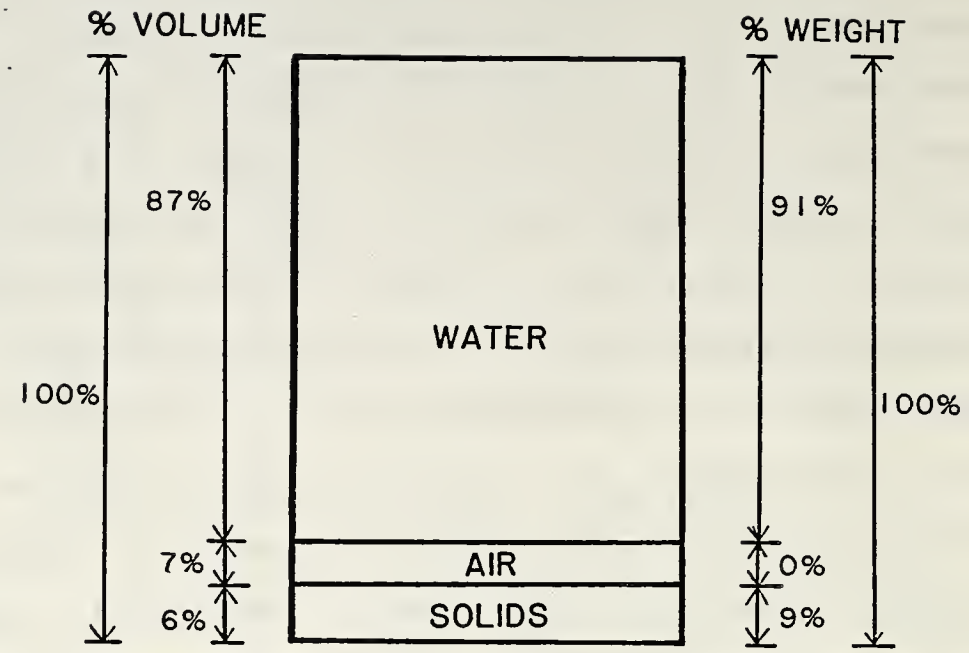
published test results indicate that peat shows an extraordinarily high increase in strength as it consolidates." It should be noted, however, it takes relatively large strains for the large increases in shear strength to develop. In general, settlements of this magnitude can not be tolerated, except by the application of a preloading method in which case most of the settlement occurs during the construction period.

Shear strength improvement. When peat is to be used as a foundation material, the mechanical properties can be greatly improved by preloading the deposit with a surcharge. Preloading would improve the engineering properties in two ways: (1) The expected settlements would be accelerated such that when the surcharge is removed the settlements under the design load would be drastically reduced, and (2) consolidating the peat would greatly increase its shear strength. Peat with its potential for large increases in shear strength with compression, seems to be idealy suited to the preloading technique. During the preloading stage, large settlements will occur, and it may be necessary to increase the surcharge height to keep the effective stress on the peat constant. The height of fill and rate of compression will depend on the stability of the surcharge load. Berms may be used to increase the resistance to slope failures.

Compressibility of Peat. Peat in its natural state has a very high compressibility when compared to nonorganic soils. This high compressibility can be attributed in part to the loose structure and the high water content of naturally occurring peats. A typical phase diagram of peat (Figure 5), shows the major constituent of peat is water. Peat in a loose state has a high permeability, thus when a load is applied, the water quickly flows out causing large volume deformations. In addition, the low density and the high void ratio of peat imply that natural peat deposits have a very small preconsolidation pressure resulting in very high volume compressibility, when a stress increase is applied.

Unfortunately, the major involvement of peat in engineering work is in its use as a foundation material. To deal with peat as a foundation material, the compression behavior must be understood and a knowledge of the pressure-settlement-time response of the soil deposit is required. In defining the settlement of peat, it is important to differentiate between deformations resulting purely from volume change and those resulting from lateral displacement (shear strain).

One Dimensional Consolidation. One dimensional consolidation of peat results in a volume change caused




Water content = 1011%

Void Ratio = 15.7

Permeability = $2 \times 10^{-4}$ cm/sec

Figure 5. Phase diagram for a fibrous Indiana Peat.

by the expulsion of water from the pores within the soil mass. If gas is present, it will also be dissipated and/or compressed. The soil particles will fill the space created by the escaping water and gas, and solid constituents will continue to adjust their relative positions during compression. In fibrous peat, during compression the fibers are reduced in size, rearranged and reoriented in a better defined manner, with faces normal to the applied load. The expulsion of water and gas occurs relatively quickly compared to mineral soils; however, the final stages of compression occur over very long periods of time. This is because the permeability of peat decreases considerably during compression, accompanied by decreasing void ratio. During consolidation of peats, the permeability can be reduced by a factor of 10,000 (Dhowian and Edil, 1980). The initially high permeability causes water pressure to dissipate rapidly. As consolidation continues, the lower permeability causes slower dissipation of pore water pressure, causing hydrodynamic effects to be more noticeable.

Ideally, consolidation of peat can be separated into four components:

1. Instantaneous strain, $\epsilon_i$
2. Primary strain, $\epsilon_p$

3. Secondary strain, $\epsilon_s$

4. Tertiary strain, $\epsilon_t$

Instantaneous strain in peat is due to elastic compression. Peat generally contains 5 to 10% gas, which contributes to large immediate initial compression and some rebound if the load is totally removed (Landva and LaRochelle, 1982). Following the immediate initial compression is a portion called primary strain. Primary strain occurs during the period of significant excess pore water pressure dissipation. This component occurs relatively quickly and amounts to a substantial proportion of total settlement. Secondary strain occurs under small to negligible pore water pressures. This portion of strain normally causes deformations which occur linearly with the logarithm of time. Secondary strain can be considered a creep phenomena in that deformation occurs under a constant effective stress. Secondary compression occurs over very long periods of time. In contrast to mineral soils, secondary compression in peat can cause a large proportion of the total settlement. Tertiary strain refers to a substantial increase in the rate of creep over that in the secondary phase. Tertiary strain has been observed only in laboratory consolidation tests (Dhowian and Edil, 1980).

These four components of consolidation are distinguishable in the results of oedometer tests at low stress levels. Dhowian and Edil (1980) conducted numerous tests on various peats and found the time-settlement curve for light loads resembles that shown in Figure 6. The secondary strain begins at $t_a$ and the beginning of tertiary compression starts at $t_k$. Dhowian and Edil (1980) went on to show that this well defined shape curve only occurred during low stress oedometer tests. When the stress became more substantial, the components of strain seemed to blend together. The vertical strain-logarithm of time curves for most of the pressure increments lacked any clear demarcation between the primary and secondary strain components (see Figure 7). Wilson et. al. (1965) suggest the rate of creep could exceed the rate of primary consolidation. The result of such a combined process could be that the primary consolidation would be masked by the creep, and the familiar "S" shape would disappear. The result of this would be that the primary strain portion could not be distinguished from the rest of the curve.

Settlement curves for embankments built over peat deposits resemble that shown in Figure 7. This seems to substantiate that when peat is loaded in the field, all components of strain are occurring simultaneously.

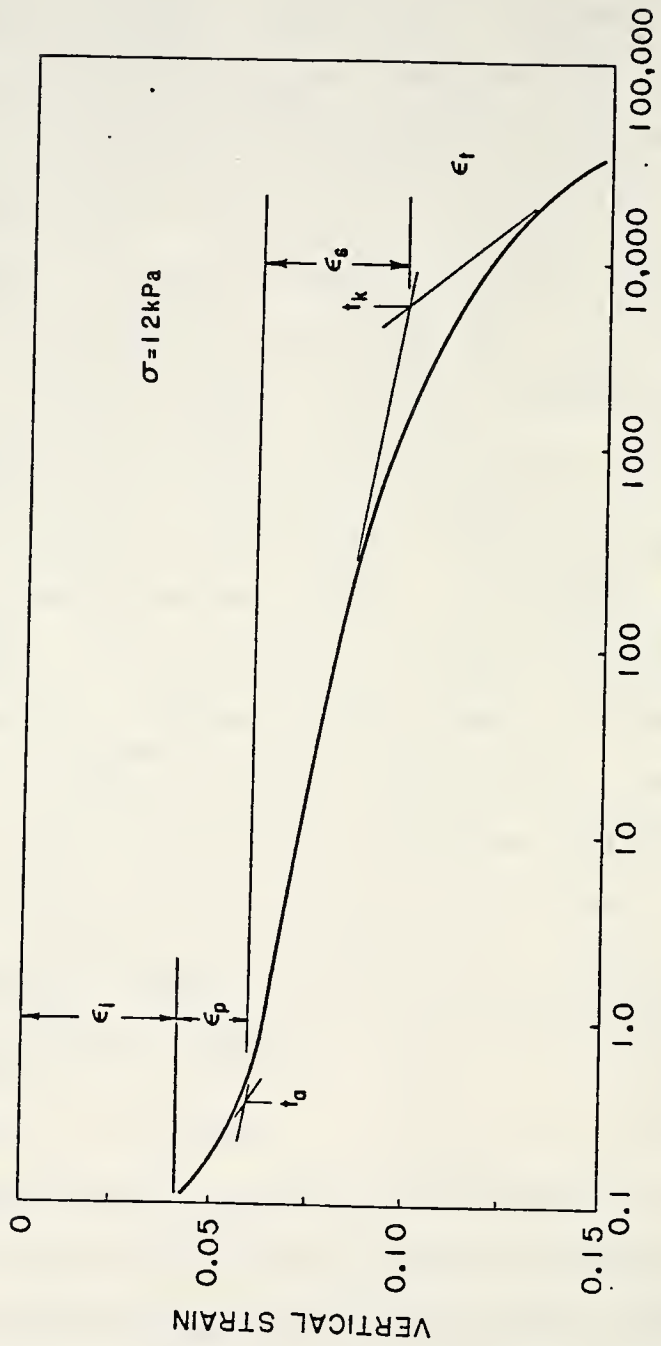



Figure 6. Laboratory Strain–Time Curve for Peat. From Dhowian and Edil (1980).

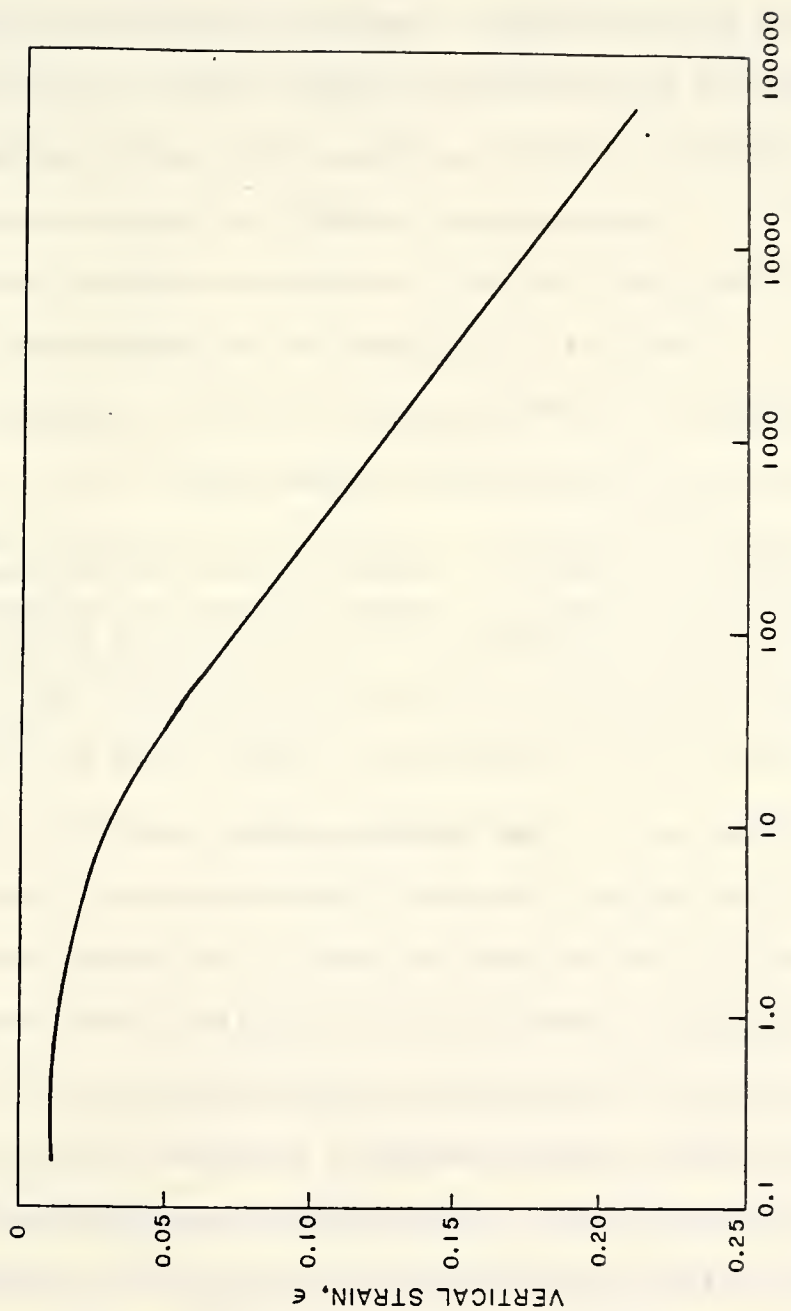



Figure 7. Consolidation response curve. From Dhowlan and Edil (1980).

Landva and LaRochelle (1982) observed that the secondary compression is of such a large magnitude, that it masks the primary portion. After the excess pore water pressure has dissipated, primary consolidation has stopped and the secondary creep continues linearly with the logarithm of time. This masking effect makes it difficult to demarcate the end of primary consolidation, which leads to difficulties analyzing the settlements. In general, the shape of the settlement versus logarithm time relationship is a function of permeability and the rate of creep (Adams, 1963).

Lateral Movements. Another mode of deformation under load is settlement accompanied by a gradual lateral expulsion of the soil from beneath the loaded area. This lateral movement results from the peat being subjected to appreciable shear stresses. These shear deformations are difficult to predict, because they can not be modeled accurately in laboratory controlled tests. Landva and LaRochelle (1982) suggest that some idea of the shear deformation could be obtained from a combination of confined consolidation tests and quick and slow unconfined compression tests. The quick tests would represent rapid field loading under partially drained conditions. The slow tests could be sufficiently slow to represent fully drained conditions. The shear deformations under completely

unconfined conditions would represent a maximum, since in the field some lateral restraint does exist.

Settlement resulting from lateral movement under the load have been found to be relatively large. Landva and Peck (1980) studied the settlements under a test embankment and concluded that 30% of the total settlement resulted from shear deformations. Helenelund (1975a) found from observations made of settlements and horizontal displacements, during and after the construction of a test embankment on fibrous sphagnum peat, that a considerable proportion of the rapid settlement during the construction period is attributable to horizontal displacements. Figure 8 shows that these shear deformations were very large initially and as a result of consolidation, diminished with time.

Factors Affecting Compressibility. The consolidation behavior of peat is influenced by many factors. The type of peat (amorphous-granular, fibrous) has a major effect on the compressibility. Amorphous granular peat compresses in a different manner than fibrous peat. As a rule, fibrous peat has higher void ratio and permeability, and consolidation proceeds more rapidly than that in humified amorphous-granular peat (Karesniomi, 1972). To illustrate this difference, Adams (1965) introduced a separate mechanism of compressibility for fibrous peat. According to this

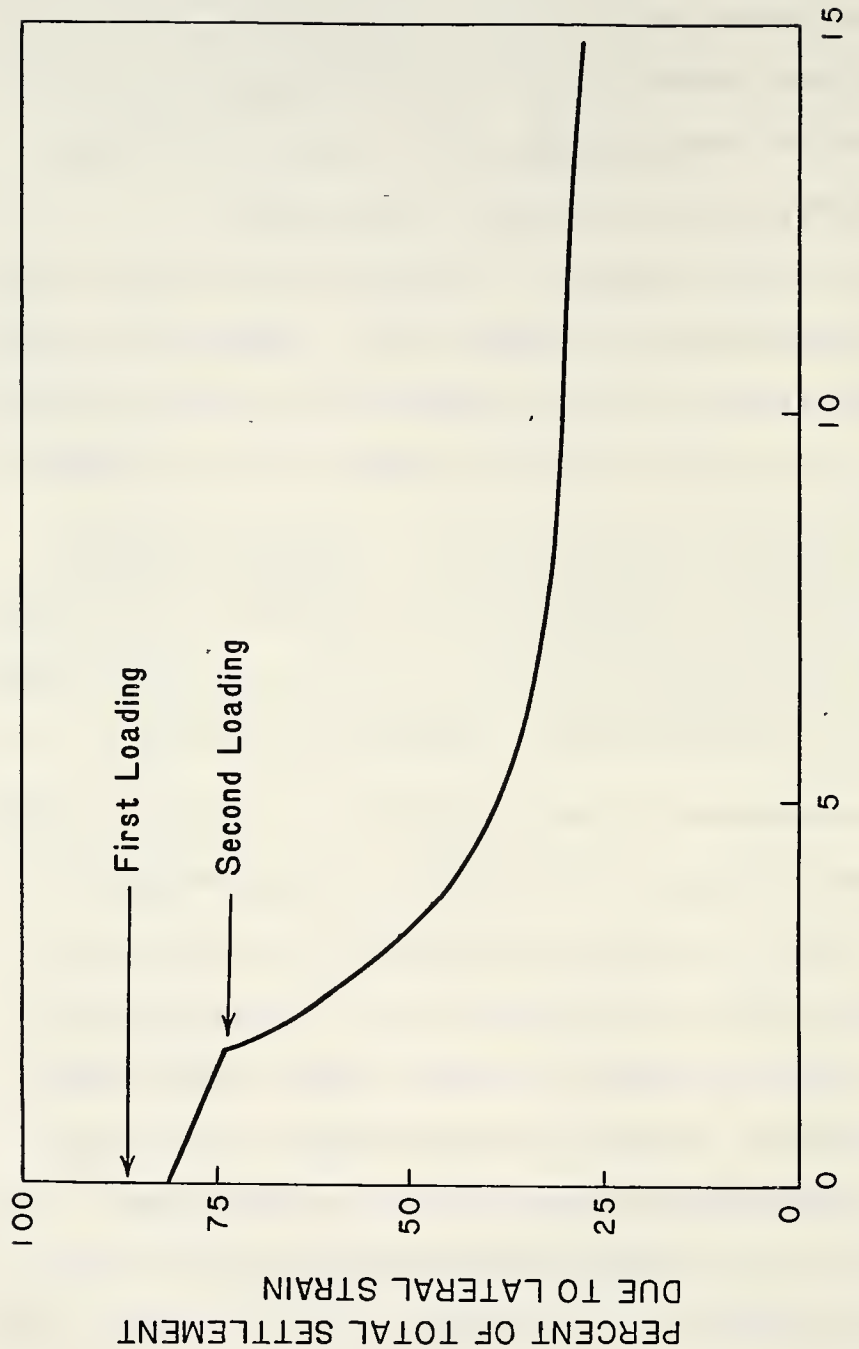



Figure 8. Results from settlement observations of a test embankment on peat. From Helenelund (1975a).

concept, fibrous peat is regarded as a system of coarse channels (macro-pores) which contain within themselves a system of compressible very fine fibers (micro-pores). The primary stage of consolidation involves the dissipation of pore-water pressure in the macro-pores. As stress is transferred to the fine fibers, water will drain from the micro-pore to the macro-pore system; this process of compression is designated as the secondary compression stage. Both stages initially occur simultaneously with the micro-pores taking much longer to compress. Dhowian and Edil (1980) suggest this may be a valid concept for fibrous peats. Berry and Poskitt (1972) also realized a difference in compressibility between amorphous-granular and fibrous peat. In developing a theory of consolidation for peat, they found it necessary to derive two separate models. One theory for amorphous-granular and a different theory for fibrous peat.

Water content and void ratio are directly proportional to compressibility. Figures 9 and 10 show suggested correlations between water content and compressibility, and void ratio and compressibility.

As discussed previously, permeability is directly proportional to compressibility. The degree of permeability dictates the rate at which water can be expelled from the peat fabric (Wilson, 1964).

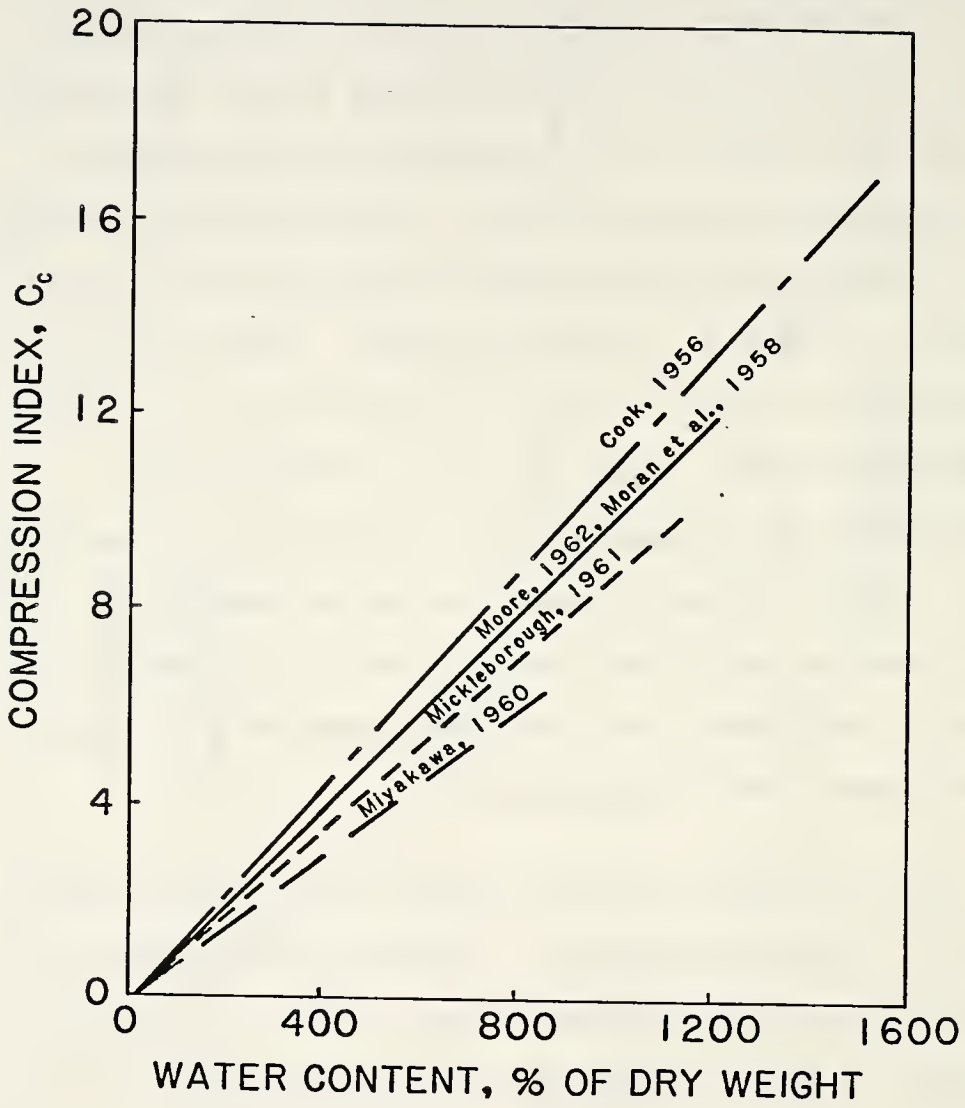



Figure 9. Compression Index versus water content. From MacFarlane (1969).

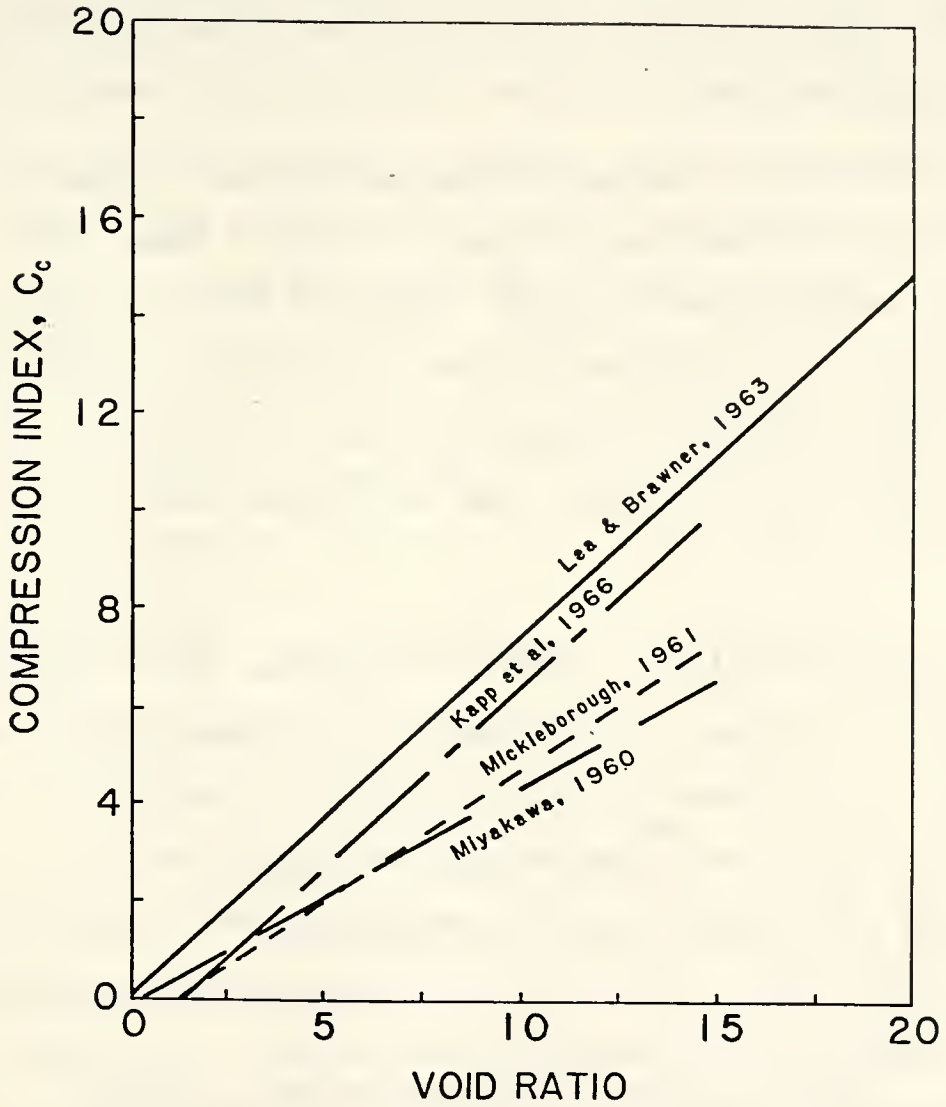



Figure 10. Compression index versus void ratio.
From MacFarlane (1969).

It has been found that mineral soil content is inversely proportional to compressibility (Anderson and Hemstock, 1959). In addition, Rutledge and Johnson (1958), found the rate of creep decreases with increasing mineral content.

Gas content affects the compressibility by reducing the area through which water can flow. Thus, gas content varies inversely with compressibility. Most peats have about 5 to 10% gas (Moran et. al. 1958, Lea and Brawner, 1963). This gas is a combination of entrapped air and gas generated by organic decomposition.

Compressibility of Natural Deposits. The depositional and physical characteristics of a peat deposit can make the estimation of settlement very difficult. A prediction of the magnitude of settlement in peat presents difficulties, mainly because of the variability of the peat. This variability can be traced partly to variations in porosity or water content, and partly as suggested by Gautschi (1965), to the fabric and structure of the peat. In addition, compressibility decreases with increasing degree of humification. Since the degree of humification often varies both horizontally and vertically, the compressibility will vary in a similar manner (Karesniomi, 1972). Due to this large and erratic variability, it is sometimes best to

predict an upper bound of the compressibility and use this value in design.

In many peat deposits, the underlying soils may be more dangerous from the standpoint of stability and settlement than the peat. Seldom is the compressible soil layer only peat. Generally it also contains organic clay and/or marl, which do not drain as quickly as peat. These materials may develop high excess pore pressures, resulting in loss of stability. If these soft layers are significantly thicker than the peat layer, they may contribute a larger portion to the total settlement than the peat. These soft layers must be considered in the design. In general, peat bog areas consist of layers of peat, organic clay, and soft normally consolidated materials with different consolidation characteristics. In consequence, correct estimation of the time-settlement behavior entails that simultaneous consolidation of two or more layer systems be taken into account.

Summary. MacFarlane (1965) summarized the knowledge as of 1965 regarding the compression and consolidation of peat. Certain conclusions on which there was then "fairly general, although not necessarily universal agreement" are shown below.

i. It was difficult to define demarcation between primary and secondary consolidation.

ii. In the laboratory, dissipation of pore pressure occurred within a few minutes, after about 50% of the total settlement.

iii. Primary consolidation ($u > 0$) was proportional to thickness, where u is the excess pore pressure.

iv. Primary consolidation rate was variously reported to be proportional to the thickness to powers of 1, 1.5 and 2.

v. Secondary consolidation ($u \simeq 0$) continued almost indefinitely.

vi. Secondary consolidation (viscous or plastic compression) was a function of temperature, nature of peat and state of stress, but was independent of drainage conditions.

vii. Secondary compression was proportional to the logarithm of time. The coefficient of secondary compression ($\alpha_s$) was a function of load history and applied load.

viii. The coefficient of permeability (k) was initially very high, but decreased rapidly with increasing load.

ix. Rebound could be very significant if load was fully removed.

x. Gas affected rate of consolidation.

xi. Vertical sand drains did not accelerate consolidation, but they might increase shear strength during construction.

Estimating settlements of peat can be very difficult. The variables are so numerous that any attempt to incorporate them in a quantitative analysis will render the procedure so involved that its practical application will be extremely difficult. When Berry and Poskitt (1972) developed their theory of consolidation for both categories of peat, despite simplifying assumptions, the solutions were given in terms of

highly nonlinear differential equations which can only be solved by resorting to the numerical approach, using a complex computer program. Until a less complicated procedure is developed, whereby the consolidation process can be described quantitatively, it might be considered more appropriate to adopt some form of empirical procedure which describes the process of consolidation as well as predicting the field settlements. In a later section, a simple theoretical approach which serves to describe the consolidation behavior and to estimate field compression will be presented. This method will quantify the consolidation process using simple and less numerous parameters. Also in a later section, theories and relationships which have been used to predict settlements of peat will be discussed.

## SAMPLING

Field sampling of peat is necessary if the properties of the peat are to be determined. Properties of interest range from those used for classification purposes to the strain-deformation characteristics. Samples may be undisturbed or disturbed depending on the intended use of the sample. In some cases, getting to the desired sampling location can be more difficult than taking the actual sample. Many peat bogs are covered with water through part of the year. Also, as a result of the low shear strength, the surface of the peat deposit may not support the weight of a vehicle. Obtaining access to the site must be considered in the site investigation plan.

Disturbed Sampling. Disturbed sampling provides peat specimens, which have been altered structurally from their initial state. Disturbed samples are normally used for visual inspection, soil classification, determination of degree of decomposition, and to determine the depth and coverage of the peat deposit. Many methods and tools are currently used to obtain disturbed samples. Samples can be taken near the surface with a shovel and at greater depths with screw augers. MacFarlane (1969) discusses the use of Hiller and Davis samplers which are used to obtain disturbed peat samples.

Undisturbed Sampling. It is desirable to obtain samples with minimal disturbance to determine fabric characteristics, natural water content, void ratio, and stress-deformation behavior. A method which has been commonly used for obtaining undisturbed samples is to cut out large block samples from a test pit. There are many drawbacks to this method. Generally only shallow samples can be taken. The sampling depth is generally limited by the high ground water level on the peat bog and the block samples are difficult to handle and transport. If the peat is fibrous, trimming the samples from the block can be very difficult. Since it is difficult to trim peat samples, the diameter of the test specimens should be equal to the diameter of the sampler. When this is done, only end trimming is required to obtain the correct size sample.

Thin walled cylindrical samplers have been used to obtain peat samples with minimal disturbance. Shelby tubes can be used near the surface. The cutting edge should be sharpened and the cylinder should be lubricated on both sides to reduce resistance while the tube is being inserted. The tube and sample are retrieved by digging the tube out with a shovel. If the shovel is placed under the tube during removal, maximum sample recovery will be obtained.

A number of different types of samplers have been developed for taking undisturbed samples from greater depths (Hvorslev 1948, MacFarlane 1969). Piston samplers constructed for taking undisturbed clay samples (SCPS, 1961) can be used in amorphous granular peat and in fibrous peat with some precompression. Landva, et. al. (1982) designed a large diameter piston sampler and a block sampler for use in peat. Another sampler developed specifically for taking undisturbed samples of organic soils is the square-tube sampler of Digerfeldt (1966). More elaborate samplers have been used, such as the Swedish metal foil sampler or by using the Dutch method where the sample is separated from the sampling tube by a plastic fluid (Begemann, 1966)

Helenelund (1972) researched the effect of several types of cutting edges used on thin walled samplers. No great differences with regard to disturbance seemed to exist between results obtained with normal plane edge or saw shape edge or other types tested. It was found that the penetration resistance could be reduced slightly by rotating the sampler in a zig-zag motion. Helenelund (1972) recommended the diameter of the cylindrical sampler should preferably be 20 cm or more to reduce side friction effects.

High quality undisturbed peat samples can be taken for research purposes by freezing the peat. During the winter when the peat is frozen is an ideal time to sample peat. The sampling site is easily accessed and the samples can be taken without compression or water loss.

Transporting, storage and sample preparation can also cause sample disturbance. Once again, frozen samples makes each of these tasks less troublesome. In spite of precautions which may be taken, deterioration of peat samples can be caused by volume distortion from the presence of gas, loss of water, or handling (Brawner, 1957). Samples in cylindrical tubes should be transported and stored with the tubes standing on their ends. The tubes should also be turned occasionally to prevent water from migrating to one end of the sample. Special laboratory equipment can be used for the trimming of the samples in preparation for testing if a truly undisturbed sample is desired. Special equipment has been developed for guided trimming and preparation of the samples for laboratory testing (Landva, 1964).

Disturbance Effects. Some disturbance almost always occurs during sampling of peat. Depending on the information required from the sample, this disturbance may or may not be significant. For example, sampling disturbance has a noticeable effect on

determination of undrained shear strength and compressibility of the natural peat deposit. On the other hand, if the drained shear strength in the sample's natural or consolidated state is required, sampling disturbance is of little effect. To consider the effect of sampling disturbance in more detail, peat will be considered in its two extreme types, amorphous-granular and fibrous.

Fibrous Peat. Generally, when high quality peat samples are required, a thin walled cylindrical sampler is used. In fibrous peat, as the sampler penetrates, the peat fibers deflect before being cut. As the fibers deflect, the peat gets compressed. This compression is facilitated by the high permeability and rapid pore water pressure dissipation characteristic of fibrous peat. Also, if the inside clearance of the sampler is small, the skin friction between the peat sample and the sampling tube causes an additional compression, not only when the sampler is pushed down into the ground, but also when the soil sample is extruded from the sampling tube. Helenelund (1972) found that a considerable compression of the peat samples take place during sampling in fibrous peat regardless of the shape of the cutting edge (saw-toothed, plane edge, etc). This results in "precompressed" peat samples giving undrained natural strength values which

are too high and coefficients of volume compressibility which are too small. In some cases the unconfined compressive strength of these samples has been 2 to 3 times greater and the coefficient of compressibility 50% smaller than the corresponding properties of the in-situ material (Helenelund, 1972). In fibrous peat, disturbance causes errors on the unsafe side. In some cases the errors can be so great that the safe bearing capacity calculated on the basis of laboratory measurements may exceed the real ultimate bearing capacity of the natural peat deposit (Helenelund, 1972).

Figure 11 shows the effect of this precompression due to sampling on compression behavior. The block samples are to represent the peat in its natural state. As seen from Figure 11 the difference between the stress-strain curves of the undisturbed block samples and those of the samples taken with thin-walled samplers is clear. If the unconfined compressive strength is assumed to correspond to an axial compression of 10%, the strength of the sample from a saw-blade sampler was more than twice the strength of a block sample.

The precompression caused by sampling operations reduces the water content of the peat sample. This can be seen in Figure 12, which shows the water content of the block sample is considerably higher than the water

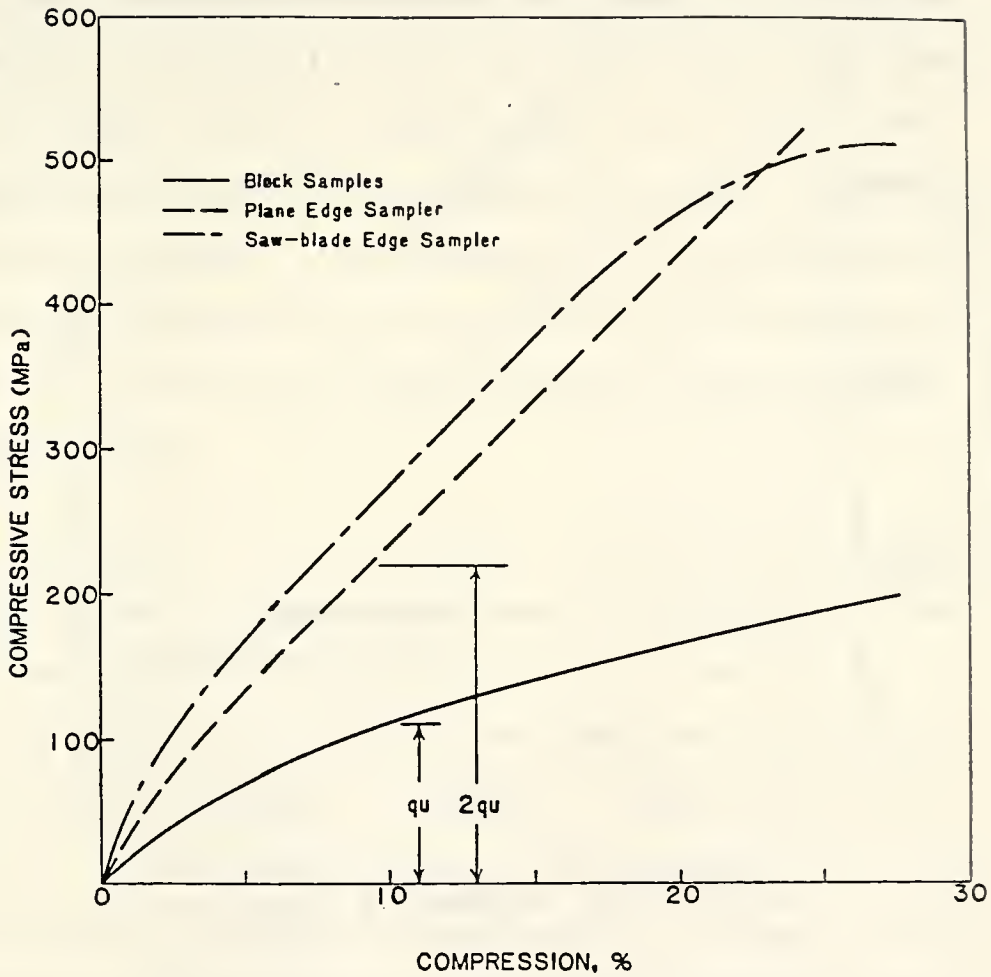



Figure 11. Results of unconfined compression tests. From Helenelund (1972a).

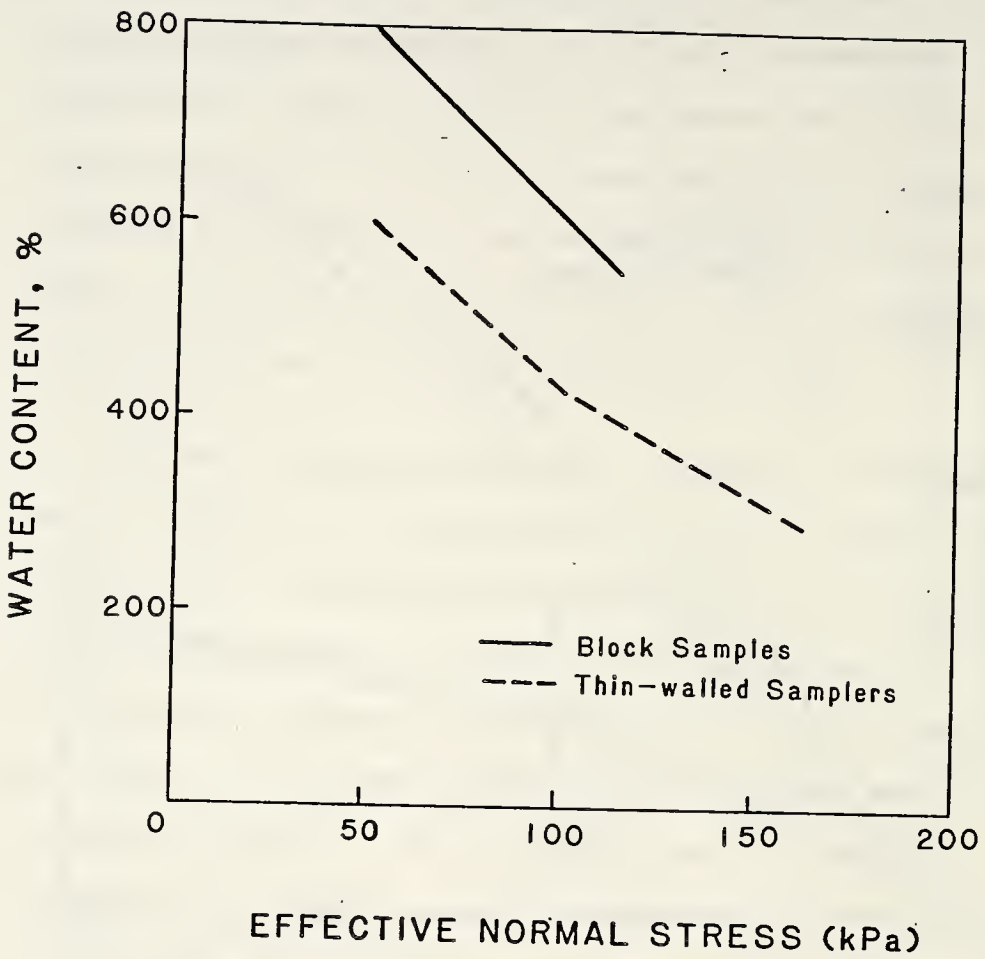



Figure 12. Variation of water content with effective stress for peat samples. After Helenelund (1975a).

content of samples taken with various types of thin-walled samplers.

Figure 13 shows the results of oedometer tests performed on peat obtained by different sampling methods. As seen from Figure 13, samples taken from samplers with normal plane and screw-shape edges have given almost identical load-compression curves. The compression of the block samples has on the other hand been much greater. The influence of the sampling disturbance (precompression) is especially noticeable during the first stage of the oedometer tests when the load is small.

Amorphous - granular peat. With increasing degree of decomposition, the influence of precompression during sampling will be smaller. This is because in highly decomposed peat, the influence of plant fibers begins to disappear. In addition, the permeability is much lower than fibrous peat. The compression and consolidation of decomposed peat occurs so slowly that the structural disturbance due to sampling cannot be compensated by strength increases due to consolidation. However, the influence of structural disturbance increases and this has an opposite influence on the measured strength and compressibility values. Amorphous-granular peat has been found to react in a similar way to a normally consolidated soft clay in

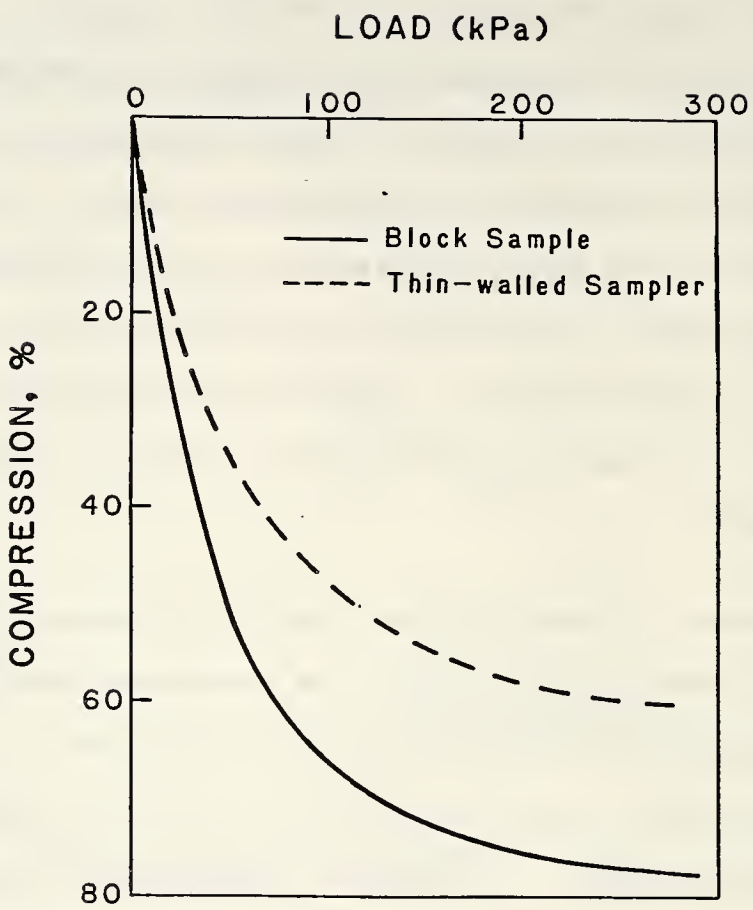



Figure 13. Load compression curves from oedometer tests. From Helenelund (1972a).

regards to sampling disturbance (Kareshiomi, 1972). In non-fibrous highly decomposed peat, the effect of sampling disturbance will cause the measured strength to be too small and the compressibility values too high. Using these values will result in a conservative design. This is just opposite of the effect of sampling disturbance on fibrous peat.

Discussion. Sampling disturbance of fibrous peat affects undrained shear strength and compressibility test values of the peat deposit in its natural state. As it turns out, these properties of the deposit are of little use in actual design. The effective overburden pressure in peat is so low and its porosity so high that even low applied loads will change the structure of the peat significantly. Peat subjected to vertical consolidation readily develops a stress induced anisotropy, where the fibers tend to align themselves horizontally. The thin layer of fill material which is generally placed over the peat deposit to provide a working surface for machinery is of sufficient weight to alter the natural peat fabric substantially. For this reason, disturbance during sampling is not so important, since the peat structure changes so drastically with compression. Landva et. al. (1982) found that practically identical shear and consolidation results were obtained for apparently undisturbed speci-

mens and specimens prepared from a completely remolded slurry. The tests were run after both samples were consolidated under equal stress levels. The test results were the same for the two samples because the structures of the undisturbed and slurried specimens were practically identical after consolidation under the initial load, as confirmed through observations under a scanning electron microscope (Landva and Pheeney, 1980).

Summary. For design purposes, the compressibility and shear strength parameters desired should be determined from samples consolidated to the same degree as the deposit, at the time the parameter is desired. For example, when a highway embankment is to be built, normally a stabilizing berm or working mat of fill is placed on the deposit. The compressibility and shear strength to be used in the embankment design should be evaluated from a sample which has been consolidated at a pressure equivalent to that existing in the field, i.e., under the weight of the existing fill plus natural overburden pressure. The small precompression due to sampling would be masked when the sample is consolidated prior to the laboratory tests. Thus, precompression caused by sampling normally does not affect the stress deformation behavior of peat samples which are to be consolidated prior to testing.

# TESTING

The testing of peat differs from mineral soils in several ways. The most obvious deviation is that when a load is applied to the peat, large deformations occur over long periods of time. Peat has a structure such that when a load is applied, the properties vary drastically. This behavior causes certain tests to give highly variable results, depending on the peat fabric and state of stress. Laboratory tests are run on peat samples for classification purposes and to give an indication of how the actual deposit will react to certain loading conditions. Although peat is an abundant material in many parts of the world, which has an important influence on land use, little has been done to compile and evaluate the many methods that have been used to determine the various properties of peat. In this section the laboratory determination of the properties which are of interest to the geotechnical engineer will be discussed.

Index Properties. The index properties are used for classification purposes and to estimate mechanical properties. ASTM (1981) has set standards for the determination of moisture, ash and organic content of peat materials (ASTM Standard D2974). Empirical correlations have been proposed relating moisture, ash and organic content to several other properties of peat.

Cook (1956) and Lea and Brawner (1963) presented the following equation which estimates the density of solids (G) from the ash content ($A_c$).

$$G = (1 - A_c)\ 1.5 + 2.7\ A_c \qquad (17)$$

Doyle (1963) has estimated that this equation can result in an error of 18 percent in an extreme case; however, these writers have found equation 17 to be adequate for practical purposes and preferable to the difficult laboratory determination test methods. MacFarlane (1969) discusses several other correlations such as density of solids and water content, void ratio and water content, bulk density and water content and shear strength and water content.

Fiber Content. The fiber content of peat is presently used for classification purposes. The fiber content has an important effect on the way peat reacts to applied stress. At this time only qualitative relations have been published relating fiber content to other physical properties of peat. Lynn et. al. (1974) recommend the following method for determining the fiber content. In this method a 5cc (cubic centimeter) syringe adjusted for a volume of 2.5cc is used to measure volumes. A 5cc plastic syringe is cut longitudinally to make the half syringe. A 2.5cc sample is measured by packing the peat into the half-syringe. The sample should be compressed just enough to force

out any entrapped air and excess water. The 2.5cc sample of peat is then transferred to a 100 mesh sieve, opening size equal to 0.147mm. The peat is gently rubbed between the thumb and finger under running water until the effluent is clear. The excess moisture is removed from the sieve by using absorptive tissues. The remainder of the sample is recollected and repacked into the half-syringe and the volume measured. This final volume divided by the initial volume represents the rubbed fiber content. These writers have found the volumes can be easily measured in a full syringe which has the constriction cut off on the end. The sample is placed into the syringe and compressed for volume measurement using the plunger. This method has been found to give consistent results on an Indiana fibrous peat.

Density. The in - situ density can be determined for peat by the same method as used for mineral soils. Care should be taken to prevent disturbance or loss of moisture during sampling.

Determination of pH. The pH of the peat deposit should be determined if materials which can be damaged by extremes in pH are to be used. An acidic peat deposit may have detrimental effects on concrete and metal which come in contact with the deposit. ASTM Standard D2976 describes the method of determining pH. The method using distilled water is of interest for

geotechnical purposes.

Compression Test. Compression testing of peat is carried out in a similar manner to mineral soils. Large strains must be anticipated, which will occur due to the nature of the peat. Trouble may occur in compression tests when using an oedometer that applies the load by use of a lever arm. This is because the large strains of the peat sample cause large movements of the lever arm which will reduce the actual weight applied to the sample. This problem can be avoided by using an oedometer that applies the load without use of a lever arm. The equipment used to measure the deformations must also be capable of measuring the large strains. The type of compression test run depends on the use of results. Single load increment and multiple increment testing is commonly used.

As part of this research, many consolidation tests were run on a fibrous Indiana peat (see Appendix C). Three types of loading sequences were used, (1) multiple increment loading, load increment ratio (LIR) equal to 1, load duration of 24 hours at each increment, (2) multiple increment loading, LIR = 1, loading only during primary consolidation (determined by pore pressure measurements), and (3) single increment loading using several samples at various stress levels, 24 hour load duration. The following conclusions were reached with

respect to compression testing of the peat used in this research.

1. Multi-step and single-step loading lead to the same amount of compression. (Landva, 1982, reached the same conclusion.)

2. Casagrande's "logarithm of time" method (Casagrande, 1938) of determining the end of primary consolidation is not satisfactory for peats due to the geometry of settlement versus time curves resulting from oedometer tests.

3. Taylor's "square root of time" method (Taylor, 1948) for determining the end of primary consolidation gave results which correlated closely with values obtained from pore pressure measurements (see figures 14 and 15).

4. All three loading sequences gave essentially the same compression index, $C_c$ (see Figure 16).

5. Multiple increment loading during the primary consolidation stage only is considered the best method for determining the compression index, $C_c$, since the test is completed in the shortest period of time.

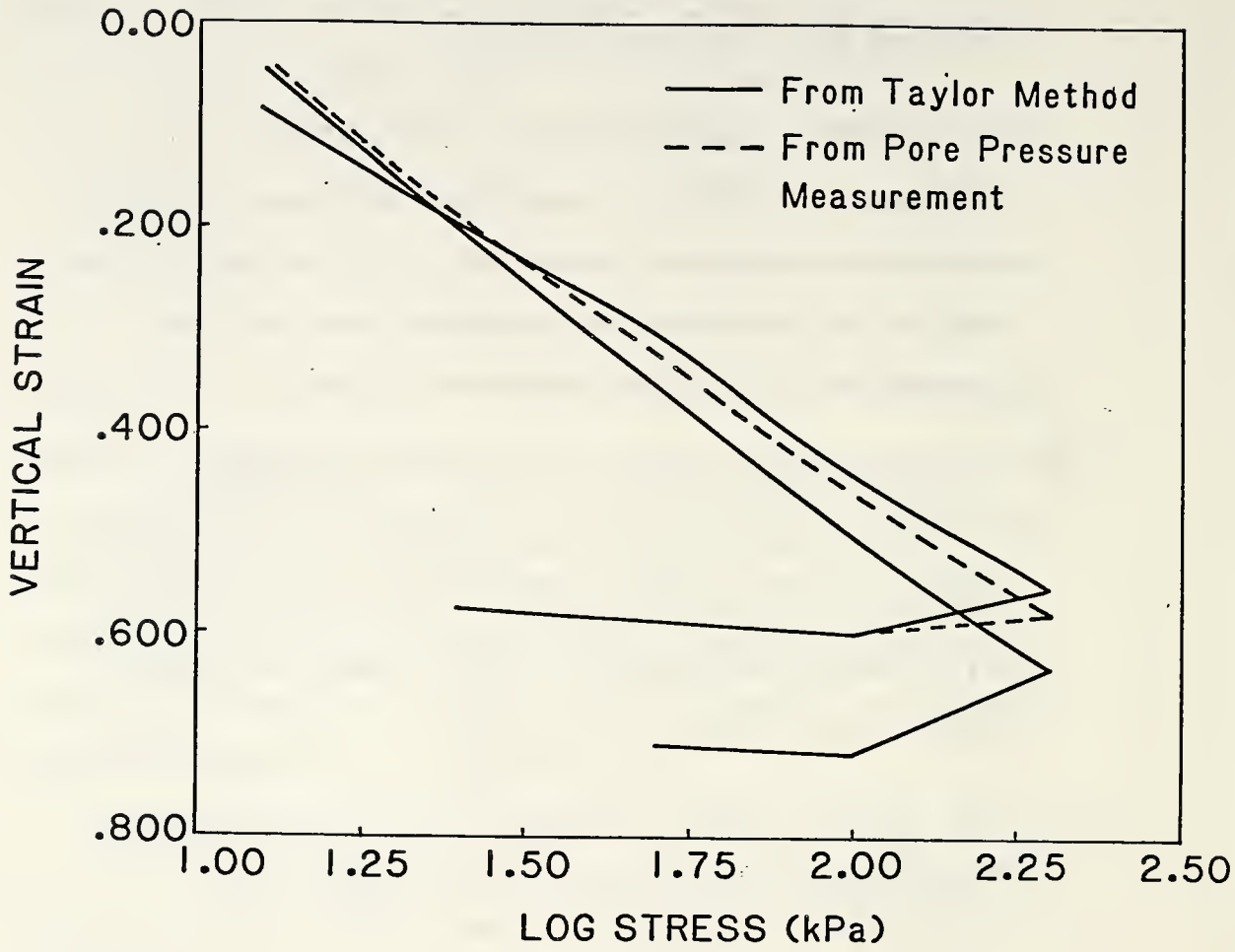



Figure 14. Comparison of Taylor's method for determining the end of primary consolidation and that obtained by pore pressure measurement. Step load test, 24 hour duration.

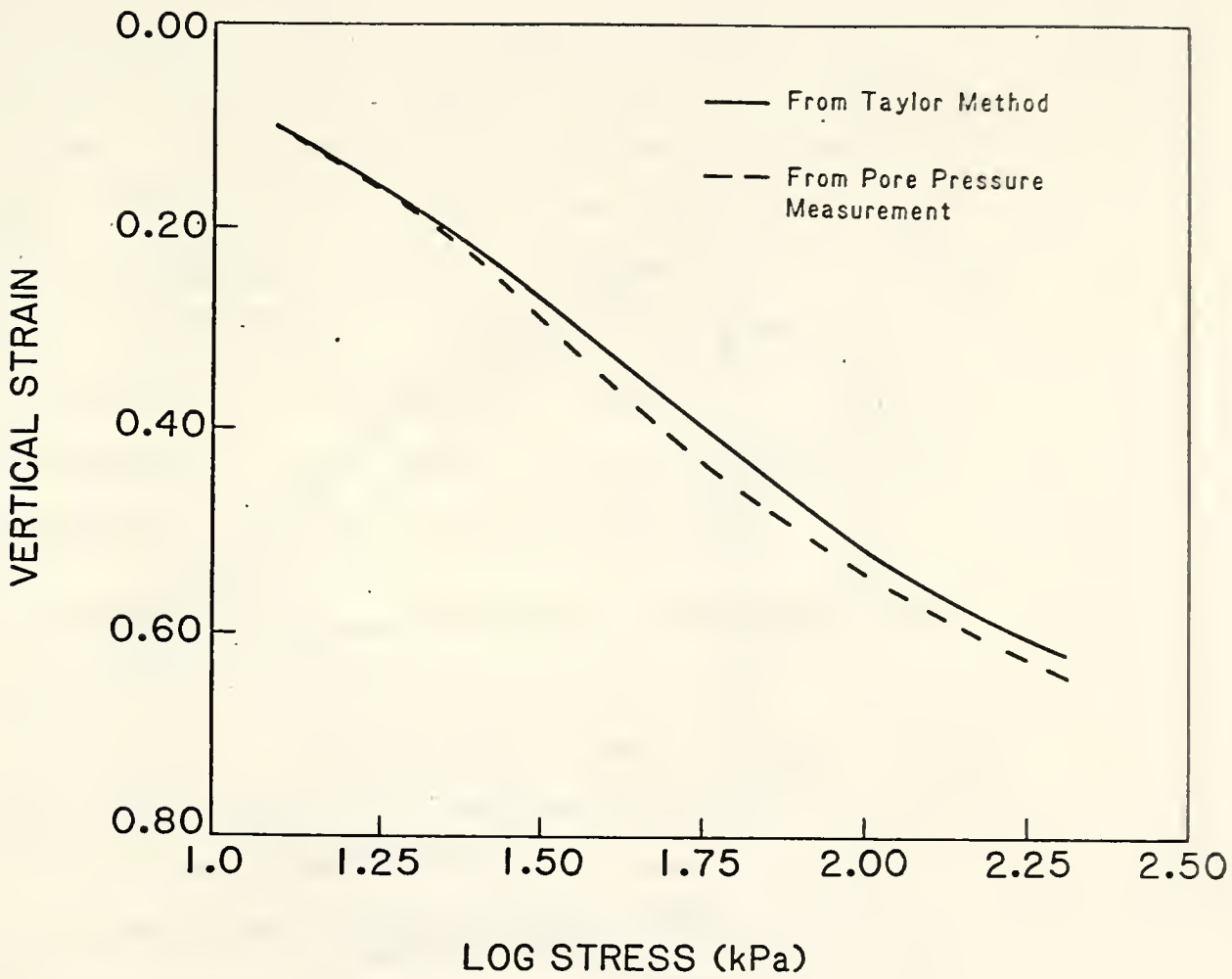
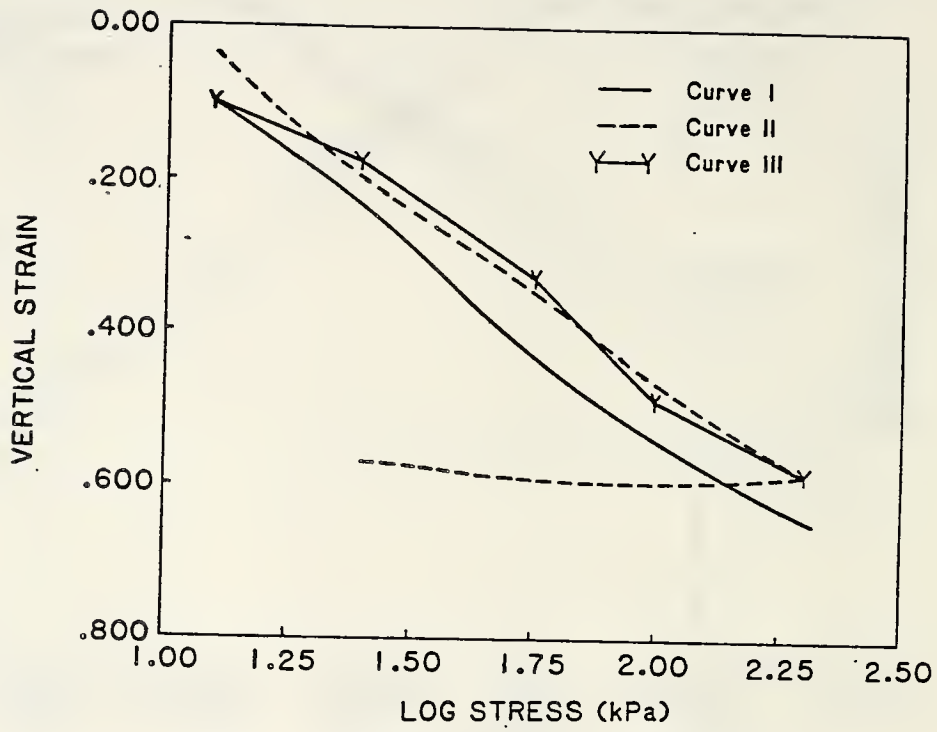



Figure 15. Comparison of Taylor's method for determining the end of primary consolidation and that obtained by pore pressure measurement. Step load test, loading during primary strain only.




Curve I – Step load test, LIR = 1, loading during primary strain only.

Curve II – Step load test, LIR = 1, 24 hour duration of each load increment.

Curve III – Single increment loading.

Figure 16. Results of oedometer tests under various loading sequences.

6. Applying the load only to the end of primary consolidation gave slightly larger strains than the other methods, thus tending to be somewhat conservative.

This writer feels that the compression samples should be back pressured to a magnitude equivalent to that existing in the field at which the depth the sample was taken. This will cause the volume of gas present in the sample to be the same as in-situ; however, the sample will not be totally saturated causing pressure transducers to give unreliable results. Keeping the fore-mentioned items in mind, the consolidation test can proceed as it would for a mineral soil.

Triaxial Testing. Once again, triaxial testing for peat is accomplished using methods similar to mineral soils. Large strains should be anticipated and failure may be caused by the sample buckling, which is not controlled by shear strength. When possible, freezing the peat sample while still in the sampling tube will minimize disturbance effects and make trimming and setting up the apparatus much easier. The type of test, unconsolidated-undrained, consolidated undrained or consolidated drained depends on the use of the results.

In the triaxial test there are two failure criteria which are generally adopted to designate the soil specimen failure. These criteria are: the bulk failure of the soil which is thought to occur at the peak value of the principal stress difference, $(\sigma_1 - \sigma_3)_{max}$, and the soil skeleton failure, which takes place at the peak value of the effective stress ratio $\frac{\sigma_1'}{\sigma_3'}$ (Holtz, 1947; Bjerrum and Lo, 1963). Dhowian (1978) found that the principal stress difference failure criterion was preferable. This is because in peat the maximum $\frac{\sigma_1'}{\sigma_3'}$ is usually reached long after $(\sigma_1 - \sigma_3)_{max}$ is attained. Dhowian (1978) added that it is easier to observe the maximum $(\sigma_1 - \sigma_3)_{max}$ during the shearing than the maximum $\frac{\sigma_1'}{\sigma_3'}$, which is a function of the developing pore water pressure.

## REVIEW OF CONSOLIDATION THEORIES

Many theoretical and empirical relationships have been developed to describe the consolidation process. Some are unique to particular soils and some are used widely on many soil types. There are two distinct processes which must be quantified by an effective consolidation model. They are: 1) the magnitude of settlement, and 2) the time rate of settlement. Consolidation is generally thought to occur in three stages, namely: 1) distortion settlement, 2) primary consolidation, and 3) secondary consolidation. In practice, deformations of the soil layer can be extremely complex. Deformations may be caused by a change of shape (distortion), or a change of volume (compression), or both. When a soil stratum is loaded, all three consolidation stages may be taking place at the same time, and separating them may be difficult.

It is a well known fact that secondary consolidation accounts for a significant amount of settlement in peat and other highly organic materials. There has been a fair amount of laboratory investigation on the physical properties of peat, and construction methods have been developed to make use of peat as a foundation material. However, there does not seem to be an acceptable correlation between laboratory and field results to determine either the magnitude or the time

rate of settlement of peat. The present methods of predicting the consolidation behavior of peat will now be examined.

Terzaghi Theory. One of the earliest models was Terzaghi's one-dimensional consolidation theory. Terzaghi (1925) used the general diffusion equation to describe the consolidation of clay in the following form.

$$c_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} \qquad (2)$$

where :

$c_v = \frac{k(1+e_o)}{a_v \gamma_w}$ = the coefficient of consolidation

$k$ = coefficient of permeability

$e_o$ = initial void ratio

$a_v$ = coefficient of compressibility

$\gamma_w$ = unit weight of water

$t$ = time

$u$ = excess of equilibrium pore-water pressure

$z$ = distance from the element to the drainage surface

In solving equation (2) the following assumptions were made: homogeneous, completely saturated medium; soil grains and water are incompressible; Darcy's law is valid; small strains; $a_v$ and k remain essentially constant and there is a unique relationship between the change in void ratio, $\Delta e$, and the change in effective

stress, $\Delta\sigma'$, (i.e., no secondary compression); and compression is one dimensional.

When equation (2) is solved, the settlement is described as a process due to the expulsion of water from the pores within the soil mass under hydrodynamic effect, and the resultant reduction in void ratio, a process known as primary consolidation.

The Terzaghi theory is simple to apply and settlements can be predicted by use of the results of one-dimensional consolidation tests. The Terzaghi theory is widely used on clays with reasonable results; however, wide variations occur between Terzaghi's theory and observed field performance when dealing with peat and other highly organic materials.

Barden (1965) examined Terzaghi's assumptions and tried to evaluate their effects when certain types of soils were used. Barden analyzed the following Terzaghi assumptions: 1) the validity of Darcy's law, 2) constant permeability and compressibility over a given pressure increment, and 3) time independence of the void ratio/effective pressure relationship.

It was determined that Item 3 is the major cause of discrepancy between the Terzaghi theory and observed behavior. Terzaghi assumed that the consolidation process is totally controlled by the dissipation of excess

pore-water pressure and that deformation will cease after pore pressure dissipates, which is not the case for many soils, particularly soils with high organic contents. Consolidation under constant effective stress (secondary consolidation) represents a large portion of the total deformation of highly organic materials, and, therefore, Terzaghi's one-dimensional consolidation theory is not applicable to peats.

Later, Terzaghi (1941) published a general hypothesis describing only qualitatively the complete continuous process of primary and secondary compression. He realized that during secondary compression, the pore-water pressure is negligible; the resistance to pressure is provided by the film and grain bonds. There is a gradual transfer of stress from film to grain bonds which is associated with very slow viscous flow or creep. This may account for the mechanism of creep in amorphous granular peat, but it is doubtful that secondary compression in fibrous peat is due completely to such a process. Terzaghi's linear consolidation theory is generally applied to peat, even though it is evident that the basic assumptions of the theory are not valid for this material. For the secondary phase, the semiempirical method of Buisman is generally used.

Buisman Theory. Terzaghi's theory proved to be acceptable for soils which exhibit only primary consolidation; however, when secondary consolidation becomes significant another model is needed. Buisman (1936) made an early attempt to provide a procedure for predicting the secondary compression of soils. He noticed that the secondary compression rate of clays was constant with respect to the logarithm of time. The Buisman expression to determine settlement is given as

$$S = h_o \sigma (\alpha_p + \alpha_s \log_{10} t) \qquad (3)$$

where :

$S$ = the settlement
$h_o$ = the initial thickness
$\sigma$ = the applied pressure
$\alpha_p$ = the primary consolidation coefficient
$\alpha_s$ = the secondary compression coefficient
$t$ = the time

It might be noticed that according to equation (3), if sufficient time passes the sample would vanish. This, of course, could not happen. The Buisman equation is widely used by the engineering profession because of its simplicity and the constant rate of secondary compression with respect to the logarithm of time is

generally encountered for most clays within the range of times considered. It was found by Dhowian (1978) that equation (3) may lead to an underestimation of settlements. Dhowian observed an increase in the rate of secondary compression, termed tertiary strain, during laboratory compression tests. Buisman's equation does not take this accelerated rate of secondary compression into account.

Berry and Poskitt Theory. Three deficiencies with the Terzaghi consolidation theory, when applied to organic soils, involve the following assumptions.

1. small strain theory
2. permeability remains constant through consolidation process
3. there exists a unique relationship between the change in void ratio and the change in effective stress (no secondary compression)

Berry and Poskitt (1972) developed consolidation theories that attempted to incorporate most of the variables encountered when dealing with peat, including finite strains, decreasing permeability and compressibility, and secondary compression time effects. Berry and Poskitt developed consolidation theories for amorphous granular peat and fibrous peat based on two rheological models representing the stress-strain-time response for both categories of peat. Both theories start with a continuity equation for an element of soil undergoing finite strain and a decreasing permeability.

The solutions to the two theories are highly nonlinear and can only be solved by use of numerical procedures and solutions involving implicit methods. It is also necessary to curve fit the secondary compression "tails" to the solution to evaluate the rheological parameters of the peat. Dhowian (1978) found discrepancies between predicted and observed compression in the laboratory. This discrepancy was attributed to the fact that the coefficient of secondary compression, $\alpha_s$ had been assumed constant with time and effective stress, which is not necessarily the case. Thompson and Palmer (1951), Lake (1961), Mesri and Godlewski (1977) and Dhowian (1978) have found that the coefficient of secondary compression is not always constant, but rather is a function of time and effective stress. Another drawback to the Berry and Poskitt theory (besides being very complex), is the assumption that there exists a linear relationship between the void ratio and the logarithm of permeability. This assumption may be acceptable, although it is not strictly valid for all soils. For compacted silts and silty clays Garcia-Bengochea, et al. (1979) found that the relationship between void ratio and the logarithm of permeability is far from linear. It must be concluded that the Berry and Poskitt theory has limited application for peat.

Gibson and Lo Theory. Gibson and Lo (1961) proposed the rheological model shown in Figure 17 to represent soils exhibiting secondary compression. This theory assumes the structural viscosity of the peat to be linear. For large values of time, the deformation behavior, $\epsilon(t)$ may be written as

$$\epsilon(t) = \Delta\sigma \left[ a+b\left(1-e^{-\left(\frac{\lambda}{b}\right)t}\right)\right] \quad , t > t_a \qquad (4)$$

Where a, b, and $\lambda$ are empirical parameters which can be determined from test data, and $t_a$ is the time after which the stress has become fully effective. Edil and Dhowian (1979) used these empirical parameters to describe the characteristic behavior of peat as follows.

$\frac{\lambda}{b}$ represents the rate of "secondary" compression (rate factor for secondary compression),

$\frac{b}{a}$ is the relative importance of the secondary to primary compression (secondary compression ratio),

$\frac{1}{\lambda}$ represents the viscosity of the soil structure during secondary compression (secondary viscosity).

This is a curve fitting method similar to Berry and Poskitt's theory but much simpler to use. Dhowian (1978) derived the following method for determining the rheological parameters a, b and $\lambda$. If equation (4) is differentiated with respect to time, the rate of strain obtained is:

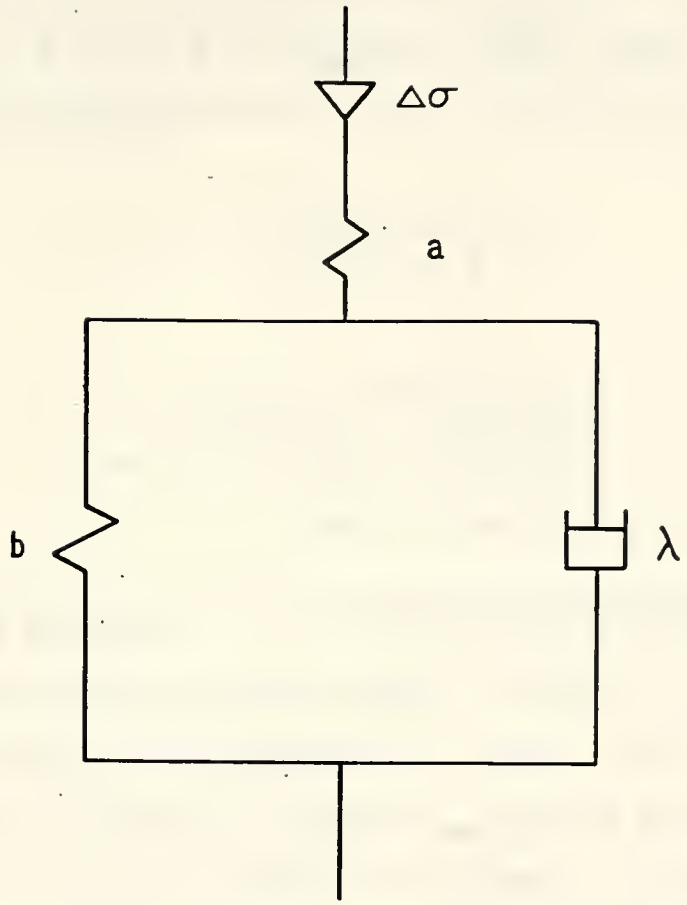



Figure 17. The Gibson and Lo model.

$$\frac{\partial \epsilon(t)}{\partial t} = \Delta\sigma e^{-(\lambda/b)t} \qquad (5)$$

Taking the logarithm of both sides in equation 5, the following linear relation is obtained:

$$\log_{10} \frac{\partial \epsilon(t)}{\partial t} = \log_{10} \Delta\sigma\lambda - 0.434 \frac{\lambda}{b} t \qquad (6)$$

Which in a simplified form is the following straight line:

$$Y = C + D(t) \qquad (7)$$

Where

$$Y = \log_{10} \frac{\partial \epsilon(t)}{\partial t} = \text{strain rate},$$

$$C = \log_{10} \Delta\sigma\lambda = \text{line intercept},$$

$$D = -0.434 \frac{\lambda}{b} = \text{slope of the line}.$$

The parameters are determined by plotting the logarithm of strain rate against time from compression results for a particular soil. A straight line is then drawn through these points. The slope (D) and the intercept (C) of this line yields the values of b and $\lambda$. The primary compressibility parameter a, is found by substituting the known quantities into equation (8).

$$a = \frac{\epsilon(t)}{\Delta\sigma} - b + be^{-(\lambda/b)t} \qquad (8)$$

Edil and Dhowian (1979) found good agreement between the Lo theory and laboratory test results, indicating that the overall shape of the long - term compression curve can be represented adequately by this rheological model.

Empirical Procedures. One method of estimating consolidation behavior of peat in the field is to run an oedometer test with the applied stress equal to that anticipated in the field (MacFarlane, 1969). The magnitude and rate of primary consolidation may be estimated as follows:

$$S_{o\ field} = \frac{H_{o\ field} \times S_{o\ lab}}{H_{o\ lab}} \qquad (9)$$

$$t_{field} = \frac{(H_{o\ field})^{i} \times t_{o\ lab}}{(H_{o\ lab})^{i}} \qquad (10)$$

Where :

$t_o$ = the time required for primary consolidation to be completed

$H_o$ = the initial peat thickness

$S_o$ = the primary compression

$i$ = the exponential parameter ranging from 1.0 to 2.0.

Secondary compression may be estimated by use of the following expression:

$$S_{s\ field} = \alpha_s H \log_{10} \frac{t}{t_o} \qquad (11)$$

Where :

$S_s$ = the settlement due to secondary compression

$\alpha_s$ = the coefficient of secondary compression

$t$ = the field time considered

$H$ = the thickness of the peat layer after primary consolidation is completed

Drawbacks of this method are:

1. it is not always possible to determine the end of primary consolidation in the laboratory,
2. it assumes the coefficient of secondary compression to be constant, which is not always the case.
3. it does not always agree with field performance.
4. The exponential parameter, i, can vary from 1.0 to 2.0, which can produce a wide range of settlement values.

Many empirical equations have been proposed relating various physical characteristics of peat to its compressibility. Table 5 summarizes many of these empirical relationships.

Table 5. Empirical equations for compressibility of peat.

| Equation | Reference |
|---|---|
| $C_c=0.0115w_n$ | Azzouz, et. al. (1976) |
| $C_c=0.086w_n$ | Maeguchi, et. al. (1965) |
| $C_c=0.013(w_n-7)$ | Kogure and Ohira (1977) |
| $C_c=0.00782w_n^{1.07}$ | Kogure and Ohira (1977) |
| $C_c=0.62(e_o-0.56)$ | Kogure and Ohira (1977) |
| $C_c=0.370e_o^{1.17}$ | Kogure and Ohira (1977) |
| $C_c=0.5(e_o-1)$ | Maeguchi, et. al. (1965) |
| $C_c=0.621\rho_d^{-1.37}$ | Kogure and Ohira (1977) |

where:

$C_c$ = compression index

$w_n$ = natural water content

$e_o$ = natural void ratio

$\rho_d$ = dry density

The compression index, $C_c$ appears in the following equation.

$$\epsilon_v = C_c \frac{1}{1+e_o} \log \frac{p_o'+\Delta p}{p_o'} \qquad (12)$$

where

$\epsilon_v$ = vertical strain

$e_o$ = initial void ratio

$p_o'$ = effective overburden pressure

$\Delta p$ = applied stress

In many peats it is difficult to establish the magnitude of $p_o'$. When dealing with a saturated peat, $p_o'$ may come very close to zero, due to the low unit weight of peat. When $p_o'$ is close to zero, the term $\frac{p_o'+\Delta p}{p_o'}$ may become very large, leading to overly large calculated strains. Landva (1980a) reported a case where the peat was "floating" in the bog, causing $p_o$ to be zero. In this case equation (12) would be inapplicable. The result of this is clear; in many cases the compression index, $C_c$ is of no use because the equation in which $C_c$ appears is undefinable when $p_o'$ equals zero, and when $p_o'$ is close to zero, the settlement estimates may be misleading.

## PREDICTING SETTLEMENTS OF PEAT

As discussed in the previous section, there are many ways of estimating settlements on peat. None of these methods as discussed thus far take into account settlements due to shear deformation. Since shear deformations can represent a large portion of the total settlement, these methods are at best only approximate estimates. Also, a peat deposit is a multiple layered system composed of various peat layers and the underlying compressible soils. An accurate prediction must take the effects of this stratification into account.

Terzaghi and Buisman methods. Terzaghi's consolidation theory in combination with Buisman's semiempirical method for the secondary phase can give approximate settlement estimates for humified amorphous-granular peat. These methods are generally applied to peat, even though it is evident that the basic Terzaghi assumptions are not valid for peat. There are other methods which are easier to apply and give more accurate estimations of peat settlement than these.

Berry and Poskitt theory. The Berry and Poskitt theory (1972) gives results as good as any other method previously mentioned. Berry and Poskitt take the important characteristics of peat settlement into account, such as finite strain, secondary compression,

variable permeability and the effect of peat type. Incorporating all these characteristics into a single theory gives a highly complex, nonlinear set of differential equations. The parameters involved in these equations are difficult to obtain, and the methods of solving the equations involve numerical procedures and other implicit methods. Berry and Poskitt's theory is suited for research; however, a less complicated method is desirable for general use.

Compression index method. A widely used method of estimating settlements is by use of the compression index, $C_c$. The compression index, $C_c$ appears in the following equation.

$$\epsilon_v = C_c \frac{1}{1+e_o} \log \frac{p_o'+\Delta p}{p_o'} \qquad (13)$$

where

$\epsilon_v$ = vertical strain

$e_o$ = initial void ratio

$p_o'$ = effective overburden pressure

$\Delta p$ = applied stress

This method does not estimate the time rate of settlement. As previously mentioned, the calculated strain is very sensitive to changes in $p_o'$ when $p_o'$ is small. In most peat deposits the initial effective overburden pressure is small due to the low unit weight

of peat. In these cases, $p_o'$ approaches zero, and the equation gives unrealistic values of strain. If the peat deposit has been subjected to a significant overburden pressure, the use of equation (13) will give more reasonable results. It should be noted that most natural peat deposits are seldom covered by more than light vegetation or water; thus, the initial effective overburden pressure is normally the same as the effective unit weight of the peat. Use of equation (13) in combination with a method for estimating secondary compression may be appropriate for rough estimations to be used in preliminary design. For a peat deposit, a statistical coverage of $C_c$ values may be obtained by correlating the compression index, $C_c$, with the natural water content, void ratio, or dry density of the peat. Table 1 previously shown presents these relations as suggested by several investigators. Once again, this is an approximate method, the smaller the initial effective overburden pressure, the larger the possible error of settlement prediction.

Empirical method. Another reasonable method of obtaining a rough settlement prediction is to scale the results of a laboratory compression test up to field conditions. The laboratory compression test is conducted with the applied stress equal to that anticipated in the field. The magnitude and rate of primary

consolidation may be estimated as follows:

$$S_{o\ field} = \frac{H_{o\ field} \times S_{o\ lab}}{H_{o\ lab}} \qquad (14)$$

$$t_{field} = \frac{(H_{o\ field})^{i} \times t_{o\ lab}}{(H_{o\ lab})^{i}} \qquad (15)$$

Where :

$t_o$ = the time required for primary consolidation to be completed

$H_o$ = the initial peat thickness

$S_o$ = the primary compression

i = the exponential parameter ranging from 1.0 to 2.0.

Secondary compression may be estimated by use of the following expression:

$$S_{s\ field} = \alpha_s H \log_{10} \frac{t}{t_o} \qquad (16)$$

Where :

$S_s$ = the settlement due to secondary compression

$\alpha_s$ = the coefficient of secondary compression

t = the field time considered

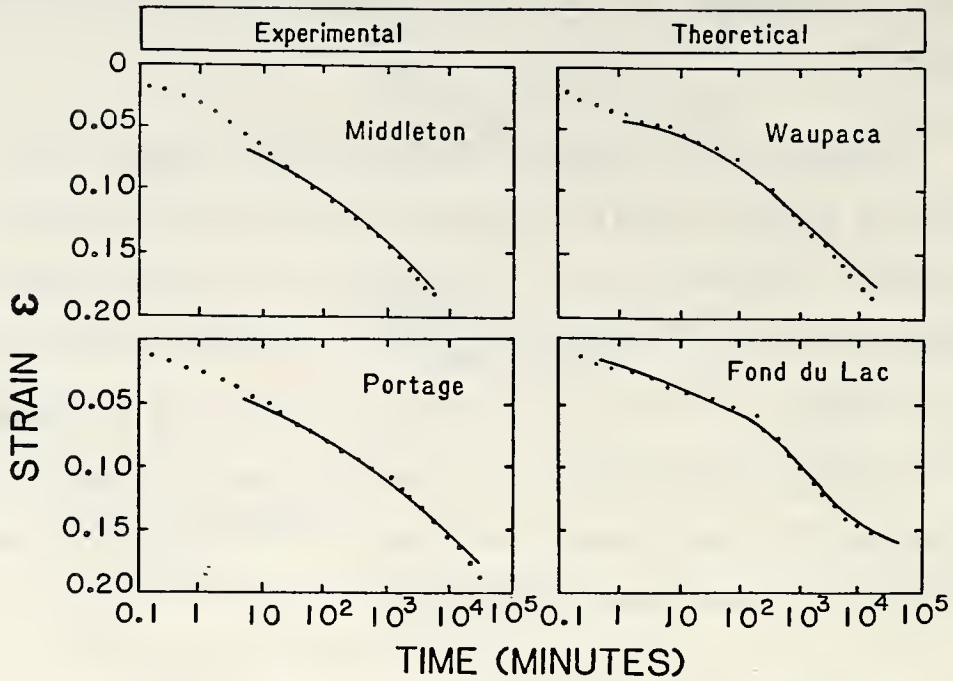
H = the thickness of the peat layer after primary consolidation is completed

This method is reasonable for preliminary estimates; however, it should be realized that discrepancies can occur. If the samples used to predict field settlements are not representative or the deposit is highly

variable, the prediction could be quite poor. In addition, the effect of underlying soils must be taken into account.

Gibson and Lo model. Gibson and Lo (1961) proposed a method similar to Berry and Poskitt's theory but much simpler to use. The details of this method are given in the previous section. Dhowian (1978) made this method even more attractive by deriving an easy method for determining the parameters which are used to estimate the settlement behavior. This method is also outlined in the previous section.

Applicability to laboratory data. Edil and Dhowian (1979) used Gibson and Lo's theory in the analysis of laboratory compression tests, and found this method modeled the shape of the compression versus time curve very closely. Relatively good agreement between the strain calculated from the model and the measured strain was obtained for four types of peat tested, ranging from amorphous-granular to fibrous in nature (see Figure 18). Compression tests performed during this research have given similar results. This seems to indicate that the model can represent the compression curves adequately as obtained from the laboratory tests.




AVERAGE PROPERTIES OF PEAT SAMPLES

| Source of Samples | Water Content (%) | Unit Weight kN/m | Ash Content (%) | Specific Gravity | Fiber Content (%) | pH | Vane Shear Strength kPa | Act-ivity | Classi-fication |
|---|---|---|---|---|---|---|---|---|---|
| Fond du Lac County, Wisconsin | 240 | 10.2 | 39.8 | 1.94 | 20 | 6.24 | 27.0 | 9.0 | Amor-phous Granular |
| Portage, Wis-consin | 600 | 9.6 | 19.5 | 1.72 | 31 | 7.30 | 14.4 | 3.0 | Fibrous |
| Waupaca County, Wis-consin | 460 | 9.6 | 15.0 | 1.68 | 50 | 6.20 | 15.0 | 3.8 | Fibrous |
| Middleton, Wisconsin | 510 | 9.1 | 12.0 | 1.41 | 64 | 7.00 | 22.0 | 4.4 | Fibrous |

Figure 18. Predicted and observed compression of peat under a pressure increment of 200 – 400 kPa. After Edil and Dhowian (1979).

Applicability to field data. In a later paper, Edil (1981) used this method to analyze the field settlements of a peat deposit under an embankment load. Using Gibson and Lo's model, projected settlements were estimated with very close correlation to the actual settlements which subsequently occurred (see Figure 19). This model was also applied to a case study presented by Samson and LaRochelle (1972). An embankment was built in three stages which applied the loading sequence shown in Figure 20 to the peat deposit. The rheological parameters were determined from the field settlement data using a computer program developed during this research (see Appendix A). The settlement behavior predicted by the model agrees quite well with the actual long term field results as shown in Figure 21. The initial discrepancy is due to the fact that the model parameters were determined using one load increment, when actually three load stages were applied in the field. This method appears to model the strain which takes place over the entire depth of the stratum. This includes the various layering within the peat and underlying soil layers. Settlements occurring from from shear strains also seem to be accounted for by this model.

As pointed out in the previous section, this model is based on rheological parameters which are obtained

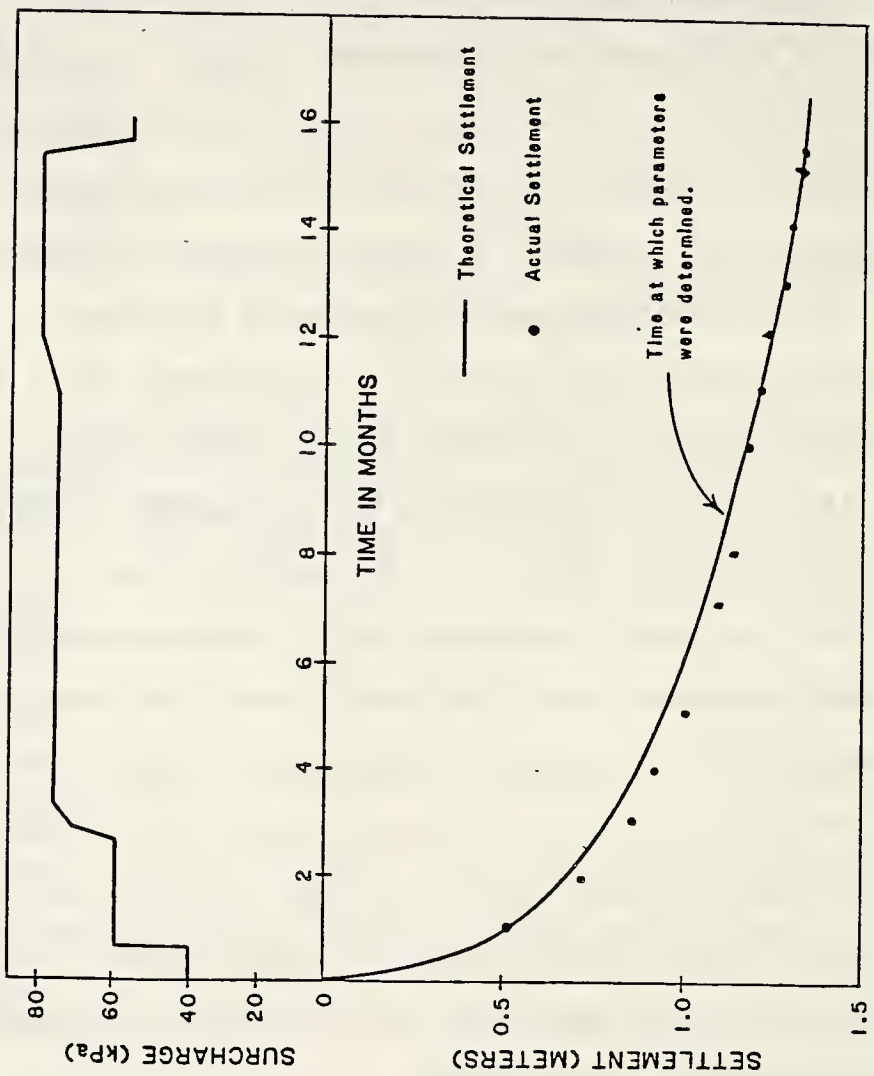



Figure 19. Settlement data under preload. After Edil (1981).

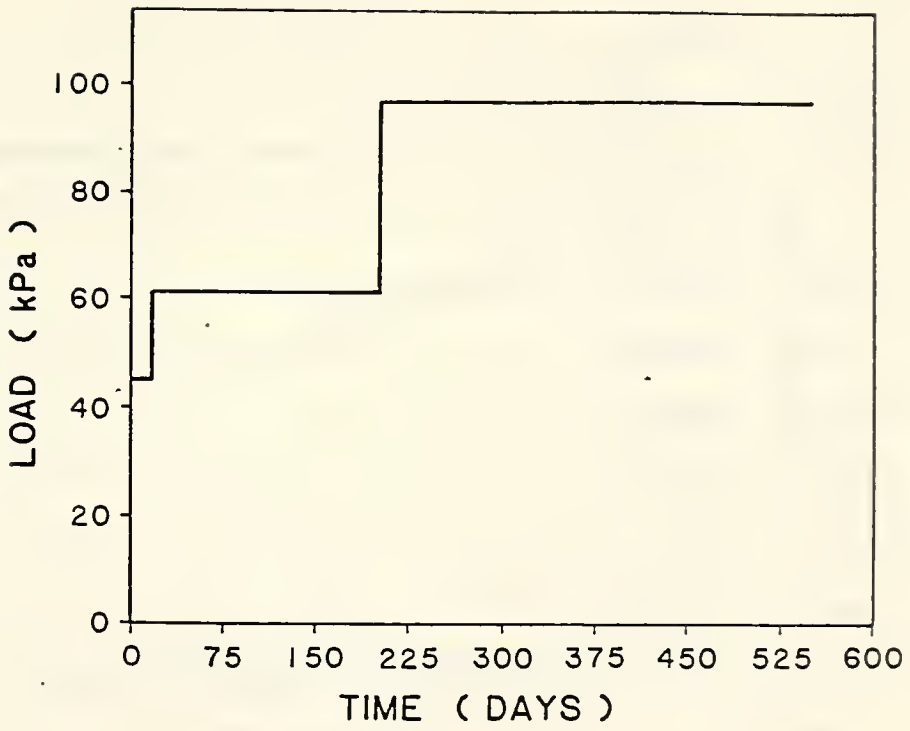



Figure 20. Loading sequence for field case.

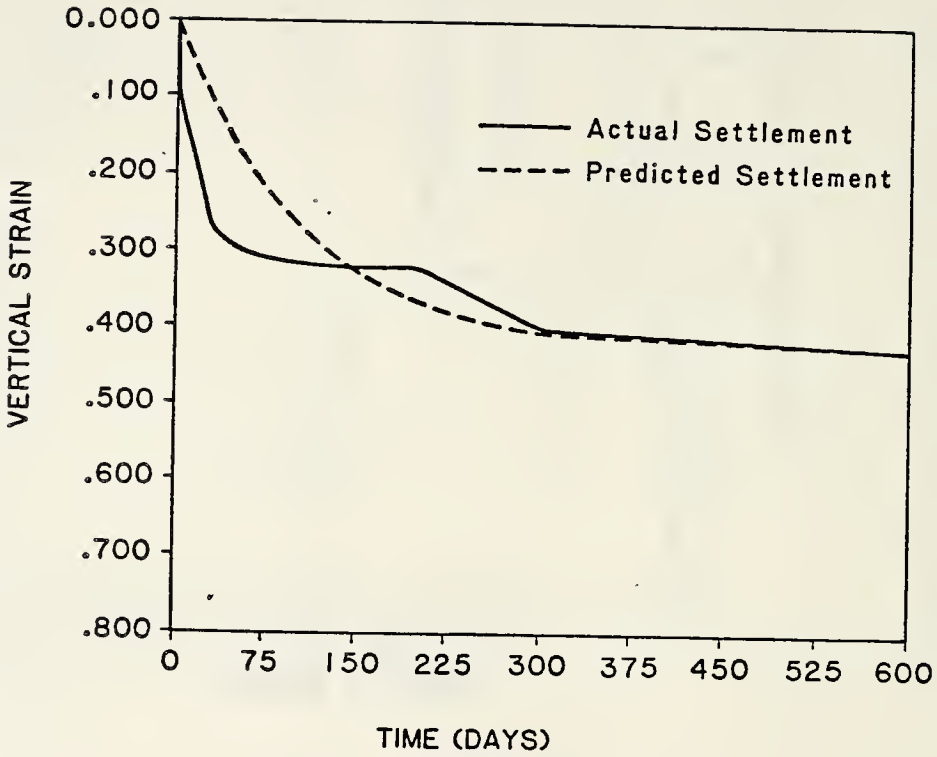



Figure 21. Application of model to field loading case.

from the actual time-settlement curve, which has progressed past the primary strain portion. Once these parameters have been obtained, the predicted settlements can be extrapolated from Equation (17), shown below, which will predict the strain at any time t, for a given stress increase ($\Delta\sigma$).

$$\epsilon(t) = \Delta\sigma\,[a + b(1 - e^{-(\lambda/b)t})] \qquad (17)$$

Varying the stress change term. Once the rheological parameters have been determined, the term $\Delta\sigma$ can be changed in Equation (17) to predict the time-settlement response of the peat deposit under a load of different magnitude. This has an application in the preloading of peat. In this application, the rheological parameters would be calculated from the settlement data resulting from the surcharged load. These parameters could be used to predict the settlements caused by the design load without the surcharge. Then the settlement curve for the design load could be compared with the settlement curve for the surcharge load so that the duration of preload would be sufficient to accelerate the expected settlements under the design load. For example, if an embankment to be built on peat causes a stress increase of 15 kPa, a total preload of 30 kPa could be applied to the deposit. The rheological parameters would be calculated from the

settlement caused by the preload (30 kPa), then the settlement caused by the design load (15 kPa) could be estimated using the parameters from the preload stage and a stress change $\Delta\sigma$ of 15 kPa using Equation (17).

Varying the stress change ($\Delta\sigma$) term in Equation (17) while using one set of rheological parameters (a, b and $\lambda$) assumes that the rheological parameters are constant with stress level and that strain is a linear function of stress at a given time. This is not the case with peat. The rheological parameters are strain rate dependent, and changes in stress do not result in a linear change in strain. However, at the stress change levels involved in preloading of peat, the violation of these assumption causes small and acceptable errors. In preloading peat, the surcharge portion of the load is seldom greater than one or at most two times the design load. Laboratory compression tests conducted during this research have shown that varying the stress change term ($\Delta\sigma$) using one set of rheological parameters, provides good settlement estimates when $\Delta\sigma_{II} \leq 2\Delta\sigma_{I}$ and reasonable estimates when $\Delta\sigma_{II} \leq 3\Delta\sigma_{I}$; where $\Delta\sigma_{II}$, is the stress level for which the rheological parameters where calculated and $\Delta\sigma_{I}$ is the stress level at which the settlement prediction is to be made. Figure 22 shows the result of using Gibson and Lo's

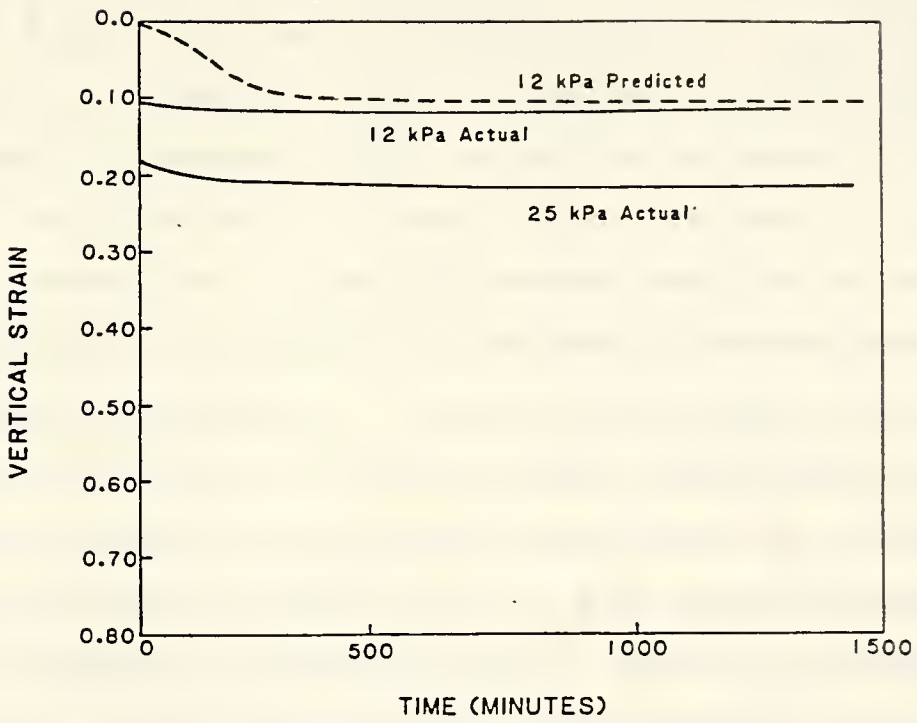



Figure 22. Use of model to predict settlement behavior of 12 kPa load using parameters from tests run at 25 kPa.

model to predict settlement behavior at a different stress level than that at which the theological parameters were calculated. Curves 1 and 2 are plots from oedometer tests on an Indiana peat subjected to stress changes ($\Delta\sigma$) of 25 kPa and 12 kPa respectively. As an illustration, curve 1 could represent the settlement recorded in the field under a preload and curve 2 could represent the settlement in the field under the design load without the surcharge. The rheological parameters were calculated from the oedometer test at 25 kPa. Curve 2 was predicted using Equation (17) with the rheological parameters from the 25 kPa test and using a stress change ($\Delta\sigma$) of 12 kPa. This predicted time versus settlement behavior under a 12 kPa load is shown by curve 3. As shown in Figure 22, the predicted settlement behavior is very close to the actual oedometer test run at 12 kPa. There is a larger discrepancy during the initial portion of the settlement curve. This corresponds to the primary strain portion of settlement which is of little concern because this phase is generally short in duration and occurs during construction of the embankment. What is of interest is the magnitude of the settlement at larger times corresponding to the secondary strain portion, and as seen in Figure 22, this model predicts the settlement past the primary strain very well. If the strains in Figure 22 were applied to a 3 meter (10 feet) thick peat stratum, the

difference between the observed and predicted settlement in this case would be 3.5 cm (1.4 in.). Figure 23 shows similar results from another laboratory test. The rheological parameters were calculated at 56.9 kPa and used to predict the settlement behavior at 25 kPa and 12 kPa. Once again the long term results are quite good. If these results applied to a 3 meter (10 feet) thick peat stratum, the difference between observed and predicted settlement would be 14 cm (5.5 in.) at 25 kPa and 10 cm (4 in.) at 12 kPa.

Case study. Edil (1981) showed that this method is valuable in analyzing the duration of preloading by an actual application. A shopping center was to be built near Madison, Wisconsin, over a peat deposit. Results of the subsurface exploration and soil properties are shown in Figure 24. The peat was preloaded and settlement data were recorded. After approximately 9 months, the parameters for use in Gibson and Lo's model were determined from the settlement data. The settlement under the preload was extrapolated using the model and compared very well with the actual settlements which subsequently occurred (see Figure 19). The slight discrepancy during the first few months is attributable to the fact that a single stress increase ($\Delta\sigma$) was used in evaluating Equation (17), when actu-

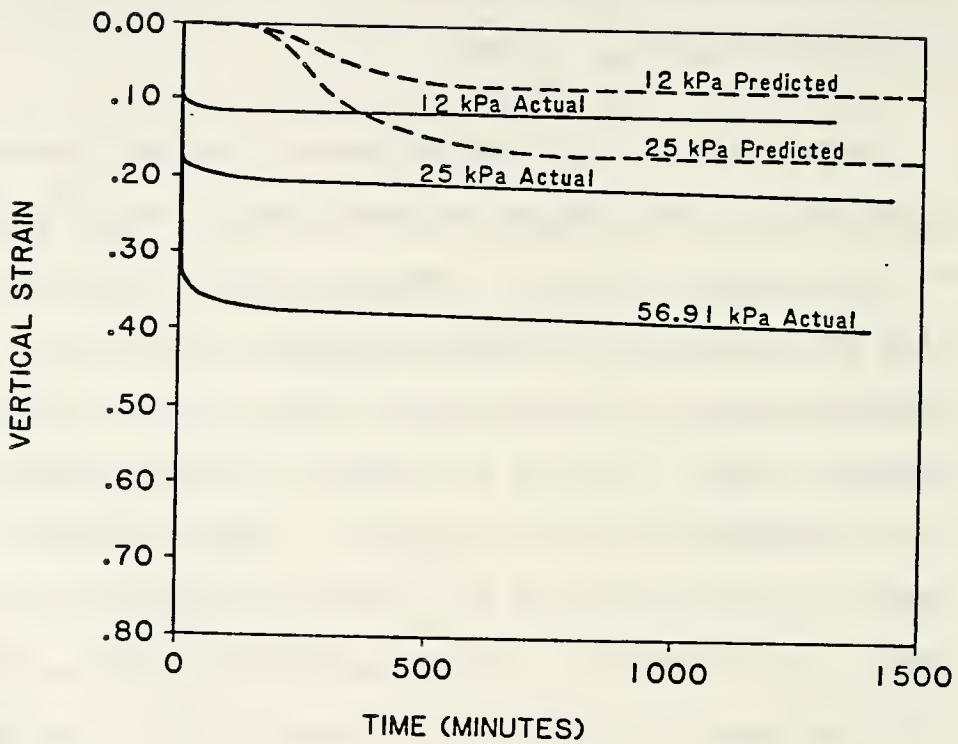



Figure 23. Use of model to predict settlement behavior of 12 and 25 kPa load using parameters from test run at 56.91 kPa.

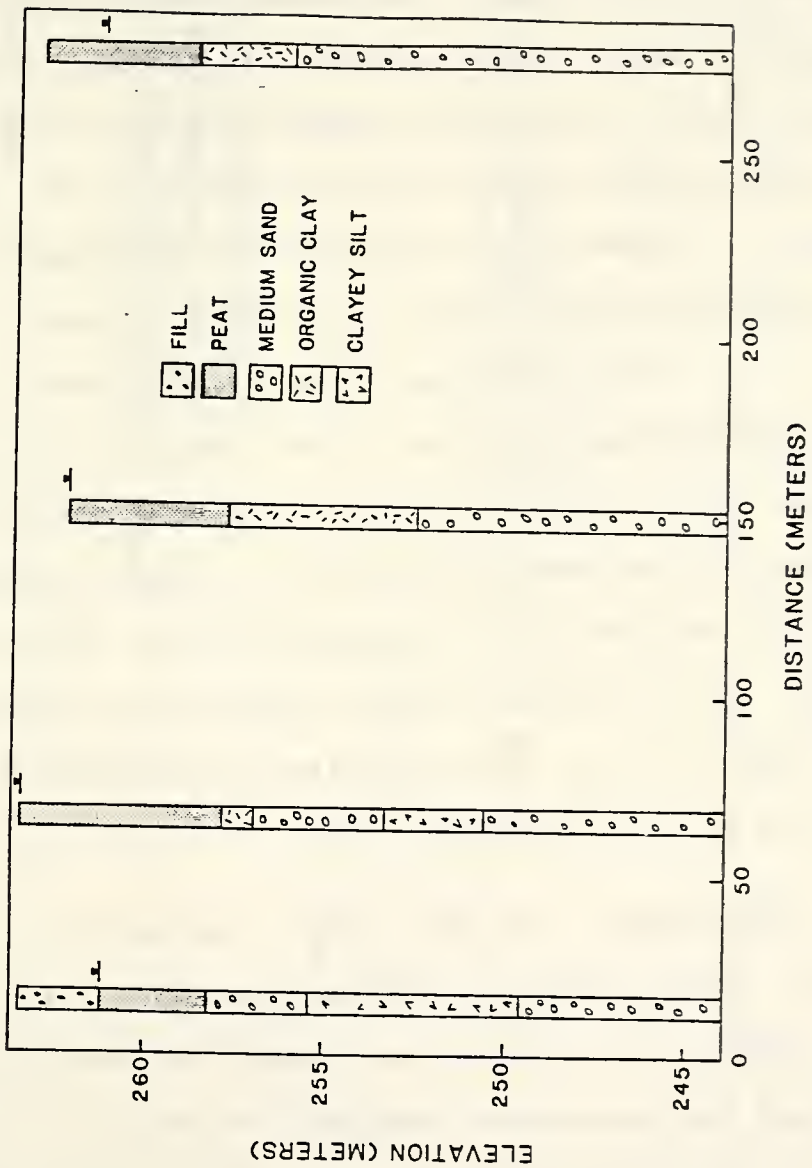



Figure 24. Longitudinal subsurface section. From Edil (1981).

ally the stress change took place in a number of steps as shown in the upper part of Figure 19. The model was applied to estimate the settlement curve for the case if the building was placed without preloading (see Figure 25). In this case, the parameters a, b and $\lambda$ were assumed to be the same as those calculated from the preload case. The stress change used was that caused by the building only. As shown in Figure 25, the surcharge was intended to eliminate the settlements expected under the load of the structure over its useful life. The theoretical building settlement curve was extrapolated to 30 years and compared with the preload curve. At that time, a marginal situation existed and the design engineer decided to postpone construction until the following spring. Edil (1981) reports that after more than 3 years since construction, there are no known problems associated with settlements of this shopping center.

Limitations. At this time it is not possible to predict field performance from laboratory testing using this method. A correlation between the parameters obtained for laboratory loading and those for field loading are yet to be developed. The problem arises from the fact that the rheological parameters used in Gibson and Lo's theory are strain rate dependent. Edil

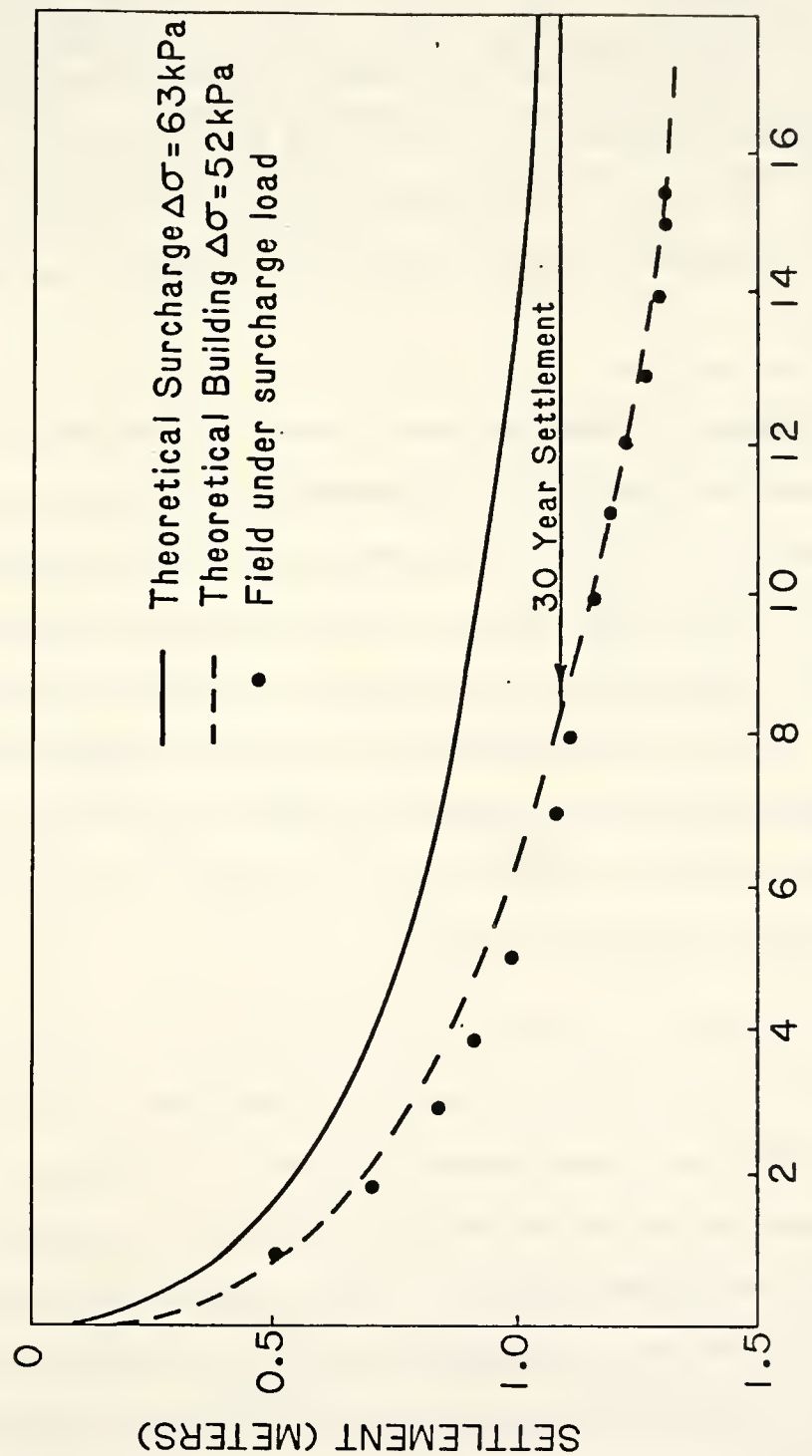



Figure 25. Theoretical predictions and field settlement data. From Edil (1981).

(1981) found that the field strain rate is often 2 to 3 orders of magnitude slower than the laboratory value during secondary compression. This difference in strain rates cause the parameters used to model the compression behavior to vary from the laboratory to the field. More data are needed relating laboratory to field parameters before a positive relation can be formed.

Summary. Reasonable settlement predictions of peat can be obtained using the methods discussed in this section. Three alternatives are recommended for rough estimates of primary consolidation, each with specific applications depending on the characteristics of the deposit. Estimates of secondary compression for preliminary analysis should be determined using either Buisman's method (equation 2) or equation (7) based on laboratory compression tests.

When peat is to be used directly as a foundation material, its properties must be improved by preloading. Using the preliminary settlement estimates, the magnitude and duration of preloading can be calculated and the surcharge applied. After the primary strain portion under the surcharge load has occurred, Gibson and Lo's theory can be applied to determine the rheological parameters used for the model. According to Landva (1980c) the field settlements under embankment

loading, have normally entered the secondary strain portion within 3 to 4 months. Knowing these parameters the surcharge settlement curve can be extrapolated, and the settlement curve for the final design load can be estimated. These two curves can be compared so that the duration of preloading is sufficient to accelerate the anticipated settlements under the final design load. Using Gibson and Lo's theory in this manner will give much more accurate control over preloading than any of the other methods. This could be considered somewhat of an observational method, in that the Gibson and Lo model gets more and more accurate as settlement continues, providing more data for determination of the parameters. The determination of the rheological parameters and settlement predictions have been simplified by use of the computer program presented in Appendix A.

## ROAD CONSTRUCTION METHODS

There are several methods of dealing with peat in relation to road construction. Many highways have been built over peat deposits using various construction methods (Brawner 1959, Lea and Brawner 1963, Mickleborough 1961, Goodman and Lee 1962, Anderson and Haas 1962, and Tessier 1966). MacFarlane (1969) has summarized some of the methods of road construction over organic terrain that has been used, as shown in Table 6. In most cases, these methods are normally narrowed to three choices for the foundation engineer to consider.

1. Relocate the road.
2. Replace the peat with a more desirable material, either by excavation and replacement or displacement.
3. Improve the properties of the peat by preconsolidation.

In many cases the choice is not clear cut. It may depend on a number of things such as time constraints and economics. This section will briefly discuss some construction methods used with peat deposits.

Relocation. One approach to highway design where organic terrain occurs has been to construct roads on stable ground around the peat deposit. Relocating the road is not always possible. In some cases the peat deposit may be so large that routing the road around it

Table 6. Road Construction Methods Over Organic Terrain. After MacFarlane (1969).

| Floating the road | Excavation, replacement or displacement | Preconsolidation | Special methods |
|---|---|---|---|
| Spread foundation (thin wide fill) | Total excavation and replacement with stable fill | Standard fill | Bridging on piles |
| Corduroy construction (fill over timber or brush) | Partial excavation and replacement with stable fill | Lightweight fill | Vertical sand drains |
| Lightweight materials (fill over peat, straw, sawdust) | Gravity surcharge method with or without water jetting | | Natural drainage |
| Reinforced mattress (fill over wire mesh war surplus landing mats) | Blasting | | Chemical stabilization |
| Lateral support (berms, rock-filled side ditches) | Underfill method | | Fill over geotextiles |
| Frozen muskeg (place fill on frozen muskeg) | Toe shooting method | | |

would not be practical. As the peripheral distance becomes larger, the incentive to cross over the peat increases. Many other factors may keep this from being an alternative.

Replacement. Thin surface layers of peat can be excavated and replaced by mineral soil. On occasion, blasting has been resorted to for the displacement of thick peat layers under embankments (Casagrande, 1966), but some peaty soil may remain in the deeper layers. Consequently, displacement methods such as blasting, toe shooting and water jetting are not recommended. Displacement by constructing a large fill over the peat, causing the material to be squeezed out under the weight of the fill is a common construction method. This is normally accomplished by end dumping material at the front end of the fill as the underlying peat is pushed aside. This method is somewhat uncertain and requires more fill material than other methods. Excavation and replacement is the most common method. When the peat is shallow, it is desirable to excavate it completely and replace it with sand, gravel or other stable materials. The minimum depth to which excavation is economical will depend on the cost of excavation, disposal, and backfill material. For deeper deposits, excavation becomes very expensive and an alternative procedure such as preconsolidation should

be considered. When soft clay underlies the peat, complete excavation and replacement of the organic material may create a critical stability problem because the load on the clay will be increased due to the larger density of the granular fill as compared to the organic material.

Preconsolidation. The technique of preconsolidation requires that a load is placed in excess of that which will be finally carried by the soft stratum and allowed to settle until the ultimate settlement that would occur under the final load has been reached. The excess load or surcharge is then removed and the construction is completed. MacFarlane (1969) mentions the following advantages preloading has over the replacement method: 1) less fill material is employed, 2) there is no peat excavation which may undermine adjacent structures, and 3) disposal areas for the excavated peat are not required.

Preloading peat. There are many cases when improving the peat as a foundation material by preloading is the most economical method. When a roadway is placed over a peat deposit, the road surface must be raised above the peat surface by means of an embankment. According to MacFarlane (1969) the minimum thickness of the embankment will depend on the loads to be imposed. 30cm (12 in.) of fill may be adequate for

automobiles, whereas 0.9 to 1.2m (3 to 4 feet) will usually be required for heavy trucks. Before construction of the embankment begins, a small fill is placed over the peat so the construction vehicles can move without getting stuck. Initially, at least 0.9m (3 feet) of fill is required in one lift to act as a working mattress for the equipment (Lea and Brawner, 1963). This lift should be compacted by several passes of a vibratory roller. The weight of this working mattress immediately begins to improve the peat as consolidation takes place. If possible, the surface mat of vegetation should not be disturbed. This surface layer adds tensile strength to the peat deposit in much the same way as an artificial fabric reinforcement would.

Geotextiles are useful in embankment construction over peat. The geotextiles will help separate the fill from the peat, control lateral spreading of the fill, and prevent local failures. According to Jarrett (1982), using geotextiles as a separator between peat and an embankment will reduce the amount of material required to construct the embankment.

The embankment is placed and compacted in small lifts to avoid local failures and cracking in the peat layer. If the embankment is to be higher than 2.4m (8 feet), berms should be constructed beside the embankment to reduce instability (Hollingshead and Raymond,

1972b). The positive effects of loading berms have been analyzed by Hollingshead and Raymond (1972b), who have recommended loading berms be constructed several days before the central embankment is started. The surcharge load is applied by increasing the height of the embankment a sufficient amount to accelerate the settlements such that they occur during the desired time interval. The surcharge load is generally one to two times the load imposed by the final embankment. For example, if a 1.2m (4 ft) embankment were to be constructed, the total embankment height during the preload stage would generally be between 2.4m (8ft) and 3.6m (12ft). The amount of surcharge depends on how quickly the preloading stage is to be completed. Conventional embankment stability analysis using undrained strengths should be used unless the loading occurs over long time periods. The preload is removed after sufficient settlement has occurred and the construction is completed. A small amount of differential settlement should be anticipated. As a result, a flexible-type asphaltic concrete surface is preferable to a rigid Portland cement concrete pavement (MacFarlane, 1969).

# SUMMARY AND RECOMMENDATIONS

Peat is abundant in many countries around the world. A geotechnical enginer must be aware of the unique behavior of peat in order to deal effectively with this organic material.

Need for a uniform classification system. Peat and highly organic soils must be separated into categories which group materials with similar behavior together. Disagreements as to the behavior of "peat," as evident from a review of the literature, generally can be shown to arise from a lack of proper definition of the materials concerned. The proposed ASTM classification of peat and organic soil (Table 3) is recommended for use. The physical description of peat for geotechnical use should include the following:

1. Fiber content.
2. Ash content.
3. Acidity.

For example, a peat with a fiber content of 50%, ash content of 10%, and pH of 5, would be designated a hemic, medium ash, moderately acidic peat.

Physical properties. The very loose structure of peat results in high water content, high void ratio,

high compressibility and low shear strength. Peat in its natural state generally has a very high permeability; however, as the peat is compressed, the permeability is drastically reduced. This behavior accounts in part for the large deviation between peat and mineral soils. The effect of the organic fibers also has an important effect on the behavior and properties of peat. Typical values of physical properties for peat are shown in Table 7.

Mechanical properties. The fiber content of peat has an important effect on shear behavior. The fibers act as a reinforcement to triaxial shear. As the peat is compressed, the shear strength increases rapidly, being a function of the friction between fibers and tensile strength of the individual fibers. Peat, with its potential for large increases in shear strength with deformation, seems to be idealy suited to the preloading technique.

Peat has a high compressibility which continues for long periods of time. Settlement of peat is due to two separate mechanisms, 1) one dimensional consolidation, and 2) lateral displacement (shear strain).

Sampling and testing. The purpose of the site investigation should be to delineate the extent and

Table 7. Typical ranges of physical properties for peat.

| | |
|---|---|
| Fiber content | 20% - 80% |
| Water content | 500% - 1500% |
| Ash content | 2% - 25% |
| Organic content | 75% - 98% |
| Void ratio | 5 - 20 |
| Density of solids | 1.4 - 2.0 |
| Natural density | 0.4 - 1.2 g/cc |
| Dry density | 0.08 - 0.32 g/cc |
| pH | 4 - 7 |
| Natural permeability | $1 \times 10^{-3} - 1 \times 10^{-5}$ cm/sec |

depth of peat coverage and to obtain samples for classification and laboratory use. The coverage of peat deposit can be obtained by geophysical methods or soundings. Disturbed samples may be obtained from split spoon samples or continuous flight augers. Reasonably undisturbed samples may be taken with thin-walled samplers such as a shelby tube near the surface, or a piston-type sampler at greater depths. Minimal sampling disturbance will occur if the sampling tubes are sharpened and lubricated. It is preferable to use a sampling tube which is the same size as the test specimen to avoid disturbance due to trimming.

The testing program should include procedures to determine the following index properties:

1. Moisture, ash and organic content (ASTM Standard D2974).

2. Density of solids (use correlation with ash content, Equation 17).

3. Fiber content (use method outlined in Testing section).

4. In-situ wet and dry density.

5. If metal or concrete is to be placed in contact with the peat, pH should be determined (ASTM Standard D2976, distilled water method only).

Embankment design. Compression tests can be run as single load or multiple increment tests, depending on the intended use of the results. For preliminary design, the compression index $C_c$ can be determined from the water content data. It is better to have a statistical coverage of $C_c$ values estimated from the water content than to run just a few laboratory compression tests to determine the compression index. The compression index can be used to get an idea of the settlement that will occur under the proposed embankment.

These writers suggest using single load tests, scaled to field dimensions using Equations (14), (15), and (16) to determine surcharge magnitude and time settlement behavior. The exponential parameter (i) can be assumed equal to 1.5 for preliminary design.

These writers recommend using consolidated undrained triaxial tests to model field behavior. The samples should be consolidated isotropically to the level of vertical effective stress anticipated in the field. Stability of the embankment need not be considered unless the height of embankment is 1m (3 feet) or more, in which case the undrained strength parameters should be used in a stability analysis. Stability analyses involving peat soils are usually made by the conventional limit design methods of comparing required stresses with available strength on potential failure

arcs, or alternatively on the basis of potential sliding blocks. Computerized stability analyses, such as those presented by Chen (1981) for three dimensional failures may be used to both refine and facilitate the computations.

If stability presents problems, loading berms should be utilized. The positive effects of loading berms have been analyzed by Hollingshead and Raymond (1972). Geotextiles also have a beneficial effect on embankment construction over peat. These fabrics placed between the embankment and peat deposit prevent local failures, inhibit pavement rutting, and prevent spreading of the embankment causing less fill to be used.

Preloading and control. If peat must be used as a foundation material, improving the properties of the deposit by use of preloading may be the most economical method. The technique presented in this report provides a tool for the designer to control the duration of the surcharge period.

During construction, settlement plates should be placed under the embankment. During and after construction of the surcharged embankment, the deformation behavior should be monitored. After the primary strain portion of the settlement has occurred (approximately

three months), Gibson and Lo's model should be applied as a control over the duration of preload. After sufficient time has passed to accelerate the desired settlements, the surcharge is removed and highway construction is completed.

Concluding remarks. Many highways have been built over peat deposits using the preload method. This report improves on previous preloading analyses by presenting an accurate method of controlling the duration of the surcharge period. During the preliminary design, a conservative estimate of the required time for surcharge is made, based on laboratory compression tests. The settlement of the field embankment should be monitored by use of settlement plates. After settlement has progressed into the secondary strain portion, Gibson and Lo's model should be applied and the required duration of surcharge determined from this data (the contractor will not object if the surcharge time can be shortened). The laboratory and field data should be compared for several embankment sections and ultimately, correlations may be determined so that the field measurements will not be routinely necessary.

The report also serves to familiarize the reader with: the relevant properties and behavior of peat, methods for building highway embankments over peat, and the latest efforts to classify peat.

Recommendations for future research. There is a need to correlate the rheological parameters from Gibson and Lo's model as determined in the laboratory to the parameters calculated under field loading situations. If this relation were determined, field settlements could be predicted from laboratory tests using this model.

The relationship between the shear strength of samples consolidated hydrostatically and those consolidated anisotropically is still not quantitatively understood. Qualitatively, the shear strength of peat is larger when consolidated by a hydrostatic pressure as compared to anisotropic, where the coefficient of lateral earth pressure at rest ($K_o$) is less than one. The magnitude of this difference has not been quantified. The actual consolidation which occurs in the field is anisotropic. To model this behavior in triaxial tests requires a knowledge of $K_o$, which is difficult to determine for peats.

Research concerning peat will be made much easier when a standard classification of peat has been adopted. At that time, when a case study is presented, the researcher will know precisely what material is being referred to. Presently this condition does not exist and causes much confusion in the literature.

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# APPENDICES

# APPENDIX A

## Computerized Gibson and Lo

## Rheological Model

```
c
c       COMPUTER PROGRAM OF THE GIBSON AND LO MODEL
c
c
c
c       This program will calculate the rheological
c       parameters for the Gibson and Lo model, and
c       generate time-settlement data using these parameters
c       for any specified stress change.
c
c
c       INPUT REQUIRED
c       line #1       stress increase causing input
c                     settlement-time data (f10.2)
c
c       line #2       stress increase to be used in prediction (f10.2)
c
c       line #3       number of input observations (I4)
c                     (settlement time input points)
c
c       line #3       maximum time desired in prediction (I4)
c
c       line #4       desired time intervals in prediction (I4)
c
c       line #5 thru #5+n--time and settlement data (f10.4,f10.9)
c
c
c
      dimension strain(25),time(25),xmid(25),xlog(25)
c
c       read in the increase in stress
c       read in the stress increase to be used in prediction (may
c       be the same as the stress increase which caused the actual
c       time settlement data being entered).
c
      read(5,9)sigma
      read(5,9)tsigma
 9    format(f10.2)
c
c       read in the number of settlement observations made,
c       the maximum time of prediction, and the time between
c       predictions.
c
      read(5,1)n
      read(5,1)ntime
      read(5,1)nspace
 1    format(I4)
c
c       read in the observed time and strain
c
      read(5,10) (time(i),strain(i), i=1,n)
 10   format(f10.2,f10.5)
      i=0
```

```
      j=n-1
c
c     calculate the logarith of strain rate and
c     the midpoint of the time interval
c
      do 15 i=1,j
      xlog(i)=alog10((strain(i+1)-strain(i))/(time(i+1)-time(i)))
      xmid(i)=(time(i+1)+time(i))/2.
 15   continue
c
c     LINEAR REGRESSION PROGRAM
c
c     taken from fortran manual, page 95
c
c
c
c     define floating point variable, an equal to n
c
      an = n
c
c     initialize all sums to zero
c
      sumx = 0
      sumy = 0
      sumxy = 0
      sumxx = 0
      i=0
c
c     the following do loop reads the input
c     values and calculates the sums
c
      do 2 i = 1,n
      x=xmid(i)
      y=xlog(i)
      sumx = sumx + x
      sumy = sumy + y
      sumxy = sumxy + x * y
 2    sumxx = sumxx + x ** 2
c
c     compute averages
c
      xav = sumx / an
      yav = sumy / an
c
c     compute corrected sums and cross products
c
      cscp = sumxy - sumx * sumy / an
      css = sumxx - sumx **2 / an
c
c     calculate fit parameters and rheological constants
c
      slope = cscp / css
      xinter= yav - slope * xav
```

```
      xlamb= (10**xinter)/sigma
      ratio=slope/-0.434
      b=xlamb/ratio
      a=(strain(n)/sigma)-b*(1-exp(-ratio*time(n)))
c
c     calculate coefficient of correlation
c
      top=0
      bottom=0
      do 3 i=1,n
      y=xlog(i)
      x=xmid(i)
      top=top+((xinter+slope*x)-yav)**2
 3    bottom=bottom+(y-yav)**2
      rsquar=top/bottom
      r=sqrt(rsquar)
c
c     write sigma, a, b, lambda and r
c
      write(6,20) sigma
      write(6,21)a
      write(6,22)b
      write(6,23)xlamb
      write(6,24)r
 20   format(5x," the increase in stress ",f10.4)
 21   format(5x," the parameter a is ",E20.5)
 22   format(5x," the parameter b is ",E20.5)
 23   format(5x," the parameter lambda is ",E20.5)
 24   format(5x," the coefficient of correlation is ",f10.4)
c
c     calculate and write predicted settlement data and
c     stress at which data was calculated.
c
      write(6,73) tsigma
 73   format(5x," the stress increase used in prediction",f10.4)
      x=0
      y=0
      do 5 i=0,ntime,nspace
      train=tsigma*(a+b*(1-exp(-ratio*i)))
      x=i
      y=train
      write(6,92)x
      write(6,93)y
 5    write(6,94)
 92   format(5x," the predicted time ",f10.4)
 93   format(5x,"the predicted strain ",f10.8)
 94   format(20x,"                    ")
      stop
      end
```

# APPENDIX B

## Peat Test Results

The peat samples used in this testing program were obtained from Noblesville, Indiana which is located approximately ten miles north of Indianapolis. The peat bog is owned by Anderson Peat and Organic Compost Company which mines the peat commercially for horticultural purposes. This deposit is drained by means of ditches excavated throughout the deposit. The surface mat of vegetation has been removed so that the top layers of peat will dry.

Samples were taken near the surface using sharpened and lubricated shelby tubes. The tubes were inserted and then dug out by hand using shovels. This method of sampling appears to result in minimal disturbance to the specimen.

The laboratory tests were run in accordance with the methods described in the Testing section of this report. The results of these tests are presented on the following pages. According to these results, this material would be classified as a hemic, medium ash, slightly acidic peat.

## Fiber Content Determination

| initial volume ml | final volume ml | fiber content % |
|---|---|---|
| 5 | 2.60 | 52 |
| 5 | 2.58 | 52 |
| 5 | 2.50 | 48 |
| 5 | 2.40 | 48 |
| 5 | 2.60 | 52 |
| 5 | 2.45 | 49 |

Average = 50%.

## pH of Peat in Water

| Sample | pH |
|---|---|
| 1 | 6.4 |
| 2 | 6.5 |
| 3 | 6.3 |

Initial pH of water = 6.75

## Loss on Ignition

| container | tare grams | oven dry weight grams | final weight grams | ash content % |
|---|---|---|---|---|
| 1 | 10.3912 | 13.8327 | 10.6294 | 6.92 |
| 2 | 10.8407 | 14.0385 | 11.0652 | 7.02 |
| 3 | 9.3431 | 13.3787 | 9.6820 | 8.40 |

## Specific Gravity of Solids Determination using kerosene displacement

| sample | initial weight grams | final weight grams | volume displaced milliliters | specific gravity |
|---|---|---|---|---|
| 1 | 44.3754 | 15.2557 | 19.1 | 1.52 |
| 2 | 43.5278 | 12.6424 | 19.0 | 1.63 |
| 3 | 49.4015 | 19.3673 | 19.5 | 1.54 |

Average = 1.56

## Specific Gravity of Solids Determined by Equation A-1

$$G = (1 - A_c)\ 1.5 + 2.7\ A_c \qquad (A-1)$$

| sample | ash content % | G |
|---|---|---|
| 1 | 0.0692 | 1.58 |
| 2 | 0.0702 | 1.58 |
| 3 | 0.0840 | 1.60 |

These values are very close to those obtained from laboratory testing. Use of the equation is recommended over the laboratory method.

## Compressibility Information

| lab. no. | $e_o$ | $w_n$ % | $\gamma_d$ pcf | $C_c$ | loss on ignition % |
|---|---|---|---|---|---|
| 55059A | 8.2606 | 408.8 | 12.3 | 4.95 | 59.8 |
| 55061B | 8.8871 | 460.4 | 11.5 | 4.6 | 55.8 |
| 55060B | 6.3861 | 295.2 | 17.7 | 3.2 | 48.9 |
| 55062B | 5.6899 | 199.6 | 20.2 | 3.5 | 64.8 |
| 55063A | 9.2434 | 434.0 | 11.8 | 5.55 | 46.7 |
| 55079B | 9.3214 | 550.7 | 10.8 | 5.1 | 43.9 |
| 55080C | 5.9806 | 248.2 | 19.5 | 2.8 | - |
| 55052B | 4.2881 | 172.9 | 22.5 | 1.85 | 48.2 |
| 55055B | 8.8997 | 472.0 | 10.3 | 5.1 | 71.0 |
| 55199B | 13.8886 | 756.6 | 6.4 | - | - |

The following empirical relations have been found to agree well with peat from Indiana Deposits.

$$C_c = 1.15 \times 10^{-2} w_n$$

$$C_c = 0.62(e_n - 0.56)$$

# APPENDIX C

## Compression Test Data

## Test Series One

one load applied to each sample for 24 hours

Sample 1
stress = 12. kPa

| time sec. | strain |
|---|---|
| 0. | 0. |
| 1. | .096 |
| 2. | .099 |
| 4. | .102 |
| 8. | .105 |
| 15. | .107 |
| 30. | .110 |
| 60. | .113 |
| 120. | .115 |
| 240. | .116 |
| 300. | .116 |
| 1080. | .116 |
| 1320. | .116 |

Sample 2
stress = 25. kPa

| time sec | strain |
|---|---|
| 0. | 0. |
| .5 | 0.170 |
| 1. | 0.173 |
| 2. | 0.177 |
| 4. | 0.181 |
| 8. | 0.185 |
| 15. | 0.189 |
| 30. | 0.193 |
| 65. | 0.199 |
| 180. | 0.206 |
| 240. | 0.208 |
| 360. | 0.210 |
| 420. | 0.211 |
| 1200. | 0.216 |
| 1440. | 0.216 |

Sample 3
stress = 56.91 kPa

| time sec | strain |
|---|---|
| 0. | 0. |
| 1. | 0.305 |
| 2. | 0.321 |
| 4. | 0.332 |
| 10. | 0.343 |
| 15. | 0.348 |
| 30. | 0.354 |
| 60. | 0.361 |
| 105. | 0.366 |
| 150. | 0.370 |
| 390. | 0.378 |
| 1080. | 0.387 |
| 1185. | 0.387 |
| 1245. | 0.388 |
| 1365. | 0.389 |
| 1395. | 0.389 |

Sample 4
stress = 100.48 kPa

| time sec | strain |
|---|---|
| 0. | 0. |
| 1. | .438 |
| 2. | .464 |
| 4. | .485 |
| 8. | .503 |
| 15. | .515 |
| 30. | .525 |
| 60. | .533 |
| 300. | .547 |
| 990. | .556 |
| 1095. | .556 |
| 1135. | .557 |
| 1275. | .558 |
| 1380. | .56 |

Sample 5
stress = 200.00 kPa

| time sec. | strain |
|---|---|
| 0. | 0. |
| 1. | .526 |
| 2. | .549 |
| 4. | .568 |
| 8. | .586 |
| 15. | .6 |
| 30. | .612 |
| 60. | .622 |
| 120. | .629 |
| 180. | .632 |
| 300. | .636 |
| 360. | .637 |
| 1140. | .643 |
| 1380. | .644 |
| 1470. | .646 |

## Test Series Two

step load test, LIR = 1, 24 hour load duration

Sample 6
load 1
stress = 12.38 kPa

| time sec. | strain |
|---|---|
| 0. | 0. |
| 1. | .044 |
| 2. | .046 |
| 4. | .048 |
| 8. | .05 |
| 15. | .051 |
| 30. | .053 |
| 60. | .054 |
| 310. | .055 |
| 1150. | .056 |

load 2
stress = 24.84 kPa

| time sec. | strain |
|---|---|
| 0. | .056 |
| .5 | .181 |
| 8. | .206 |
| 15. | .210 |
| 30. | .213 |
| 60. | .216 |
| 210. | .222 |
| 390. | .224 |
| 1470. | .228 |

load 3
stress = 49.61 kPa

| time sec. | strain |
|---|---|
| 0. | .228 |
| 1. | .316 |
| 2. | .323 |
| 4. | .329 |
| 8. | .335 |
| 15. | .340 |
| 30. | .345 |
| 60. | .350 |
| 210. | .357 |
| 525. | .364 |
| 1410. | .370 |

load 4
stress = 99.16 kPa

| time sec. | strain |
|---|---|
| 0. | .370 |
| 1. | .440 |
| 4. | .457 |
| 8. | .464 |
| 15. | .469 |
| 30. | .475 |
| 60. | .480 |
| 155. | .487 |
| 1440. | .505 |

load 5
stress = 198.46 kPa

| time sec. | strain |
|---|---|
| 0. | .505 |
| 1. | .542 |
| 2. | .550 |
| 4. | .559 |
| 8. | .567 |
| 15. | .574 |
| 30. | .580 |
| 60. | .586 |
| 210. | .596 |
| 540. | .602 |
| 750. | .605 |
| 1395. | .608 |

unload 6
stress = 99.16 kPa

| time sec. | strain |
|---|---|
| 1. | .602 |
| 2. | .601 |
| 4. | .601 |
| 8. | .600 |
| 15. | .600 |
| 30. | .599 |
| 180. | .598 |
| 360. | .598 |

unload 7
stress = 49.61 kPa

| time sec. | strain |
|---|---|
| 0. | .598 |
| 1. | .594 |
| 2. | .593 |
| 4. | .592 |
| 8. | .591 |
| 15. | .590 |
| 30. | .589 |
| 220. | .584 |

unload 8
stress = 24.84 kPa

| time sec. | strain |
|---|---|
| 0. | .580 |
| .5 | .579 |
| 1. | .578 |
| 2. | .577 |
| 4. | .577 |
| 20. | .575 |
| 80. | .573 |
| 12.38 | |

## Test Series Three

step load test, LIR = 1, 24 hour duration

sample 7

load 1
stress = 12.38 kPa

| time sec. | strain |
|---|---|
| 0. | 0. |
| 1. | .09 |
| 2. | .094 |
| 4. | .096 |
| 8. | .099 |
| 15. | .101 |
| 30. | .103 |
| 60. | .106 |
| 240. | .123 |
| 360. | .124 |
| 1440. | .125 |

load 2
stress = 24.76 kPa

| time sec. | strain |
|---|---|
| 0. | .125 |
| .1 | .160 |
| 2. | .212 |
| 4. | .222 |
| 8. | .23 |
| 15. | .236 |
| 30. | .242 |
| 60. | .247 |
| 120. | .252 |
| 180. | .254 |
| 1200. | .263 |
| 1440. | .265 |

load 3
stress = 49.53 kPa

| time sec. | strain |
|---|---|
| 0. | .265 |
| 1. | .347 |
| 2. | .365 |
| 4. | .382 |
| 8. | .398 |
| 15. | .410 |
| 30. | .421 |
| 60. | .428 |
| 120. | .436 |
| 180. | .439 |
| 240. | .442 |
| 1440. | .458 |

load 4
stress = 99.09 kPa

| time sec. | strain |
|---|---|
| 0. | .458 |
| 1. | .493 |
| 2. | .505 |
| 4. | .519 |
| 8. | .534 |
| 15. | .548 |
| 30. | .563 |
| 60. | .575 |
| 120. | .586 |
| 1110. | .610 |
| 1320. | .612 |
| 1440. | .612 |

load 5
stress = 198.36 kPa

| time sec. | strain |
|---|---|
| 0. | .614 |
| 1. | .633 |
| 2. | .638 |
| 4. | .645 |
| 8. | .654 |
| 15. | .664 |
| 30. | .678 |
| 60. | .692 |
| 300. | .715 |
| 1190. | .726 |
| 1340. | .726 |
| 1460. | .727 |

unload 6
stress = 99.09 kPa

| time sec. | strain |
|---|---|
| 0. | .727 |
| 1. | .723 |
| 5. | .721 |
| 15. | .719 |
| 30. | .718 |
| 60. | .717 |
| 120. | .717 |

unload 7
stress = 49.53 kPa

| time sec. | strain |
|---|---|
| 0. | .716 |
| 1. | .713 |
| 5. | .711 |
| 15. | .709 |
| 30. | .708 |
| 750. | .702 |
| 810. | .701 |

unload 8
stress = 0 kPa

| time sec. | strain |
|---|---|
| 0. | .701 |
| 5. | .692 |
| 15. | .687 |
| 30. | .682 |
| 60. | .676 |
| 180. | .664 |
| 270. | .658 |
| 315. | .656 |

## Test Series Four

step load test, LIR = 1, primary loading only

sample 8

load 1
stress = 12.38 kPa

| time sec. | strain |
|---|---|
| 0. | 0. |

load 2
stress = 24.76 kPa

| time sec. | strain |
|---|---|
| 0. | .1 |
| .1 | .198 |

load 3
stress = 49.52 kPa

| time sec. | strain |
|---|---|
| 0. | .232 |
| 1. | .374 |
| 2. | .387 |
| 4. | .396 |
| 7. | .402 |

load 4
stress = 99.07 kPa

| time sec. | strain |
|---|---|
| 0. | .402 |
| 1. | .483 |
| 2. | .500 |
| 4. | .515 |
| 9. | .528 |
| 15. | .534 |
| 30. | .541 |
| 37. | .543 |

load 5
stress = 198.37 kPa

| time sec. | strain |
|---|---|
| 0. | .543 |
| 1. | .563 |
| 2. | .573 |
| 4. | .589 |
| 8. | .602 |
| 35. | .625 |
| 65. | .631 |
| 250. | .641 |

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